TWO SECTIONS—SECTION 1

Engineering News

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Contents, First Page
Advertising Index, Last Page

UNIVERSAL PIPE
for SEWAGE DISPOSAL SYSTEMS

Sewage Disposal Plant
Oakland, N.J.

TANK—Connection from Taylor Syphon to sprinkling filter is UNIVERSAL PIPE, reducing in sizes from 14 inch to 6 inch. A 12X10 double Y provides for branch lines to different sections of the filter

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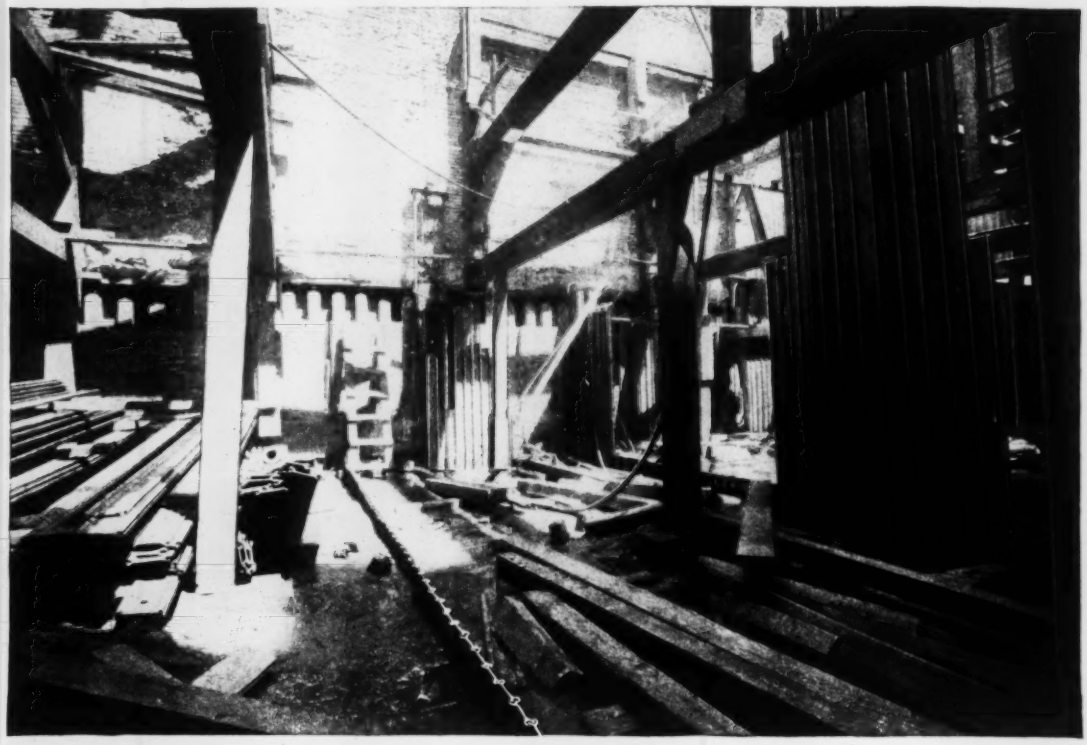


Lackawanna Steel Sheet Piling Used in Excavation Work on the Union National Bank Building, Cleveland, O.

Crowell, Lundorf-Little Co., Contractors, Cleveland.

This 17-story building, on Main Street, has paved alleys on one side and back and a massive 6-story brick building on the other side, and is located over the typical sub-strata of quicksand and hardpan which underlie the business district of Cleveland.

Construction of the foundation therefore required that all material underlying the street, alleys and adjoining building should be held in position to prevent any settlement that might cause unforseen damage to the building, paving, water pipes, etc.

The excavation had to be carried to a lower level than the foundation of adjoining buildings, so 14-in. Lackawanna Steel Sheet Piling was driven along three sides of the site, and braced across the entire width of the excavation by heavy timbers held by jacks against waling pieces. Incidentally, the steam pile driving hammer with false leads of pipe, which are shown in the shadow of some of the cross bracing offer a good method of driving steel sheet piling close to adjacent buildings.

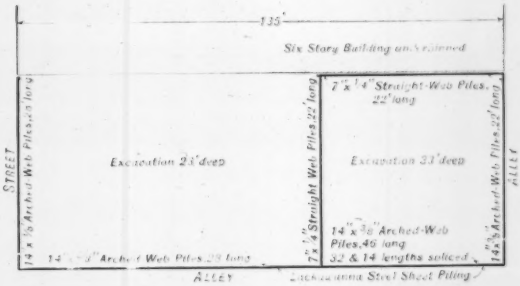
After this heavy sheet piling had been driven, a line of 7-in. Lackawanna Steel Sheet Piling was driven across the site, between the deep section and the shallow one. At the same time the wall of the adjacent building near the deep section was underpinned and more 7-in. steel sheet piling was driven here for added protection.

As excavation proceeded in the deep section, 7-in. Lackawanna Steel Sheet Piling was driven to form three cofferdams, part of which are shown above. These provided additional protection against possible failure of steam and water mains, and helped to prevent seepage of water and sand into the excavation.

The smaller Lackawanna Sheet Piling section was also utilized in a circular construction which formed a sump pit.

All of this Lackawanna Steel Sheet Piling has now served its purpose and the performance was satisfactory in every particular.

Write for our book "Lackawanna Steel Sheet Piling" and if you want it, the free advice of our Steel Sheet Piling Engineers.



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247

Engineering News

VOLUME 76 JULY 13, 1916 NUMBER 2

Tests of Large Bridge Struts Reveal New Facts

Important new facts about columns have been learned from the results of tests on eighteen very large bridge columns that were tested to failure on the 5,000-ton testing machine of the United States Bureau of Standards at Pittsburgh. Abnormal bridge steels were represented in the tests, ranging from hard or high-carbon steel to silicon steel, the most recent bridge material. The results stand out prominently as a further demonstration of the conclusion drawn from previous tests of large columns, namely, the failure point is at or very near the yield point of the metal. An absolute check on this fact is not obtainable because the Bureau investigated the metal only for its tensile yield point, and did not determine the compressive properties. The detail construction of the columns was found to have so marked an influence on the manner of failure that this feature dominated the tests. Only twelve out of the eighteen test columns showed integral bending; the other six failed by local crippling. Unsuspected weaknesses in pin jaws and latticing were revealed. A full report on the tests, prepared by J. H. Griffith and J. G. Bragg, has been submitted to the Director of the Bureau of Standards. The present notes are from a very brief summary of the report.

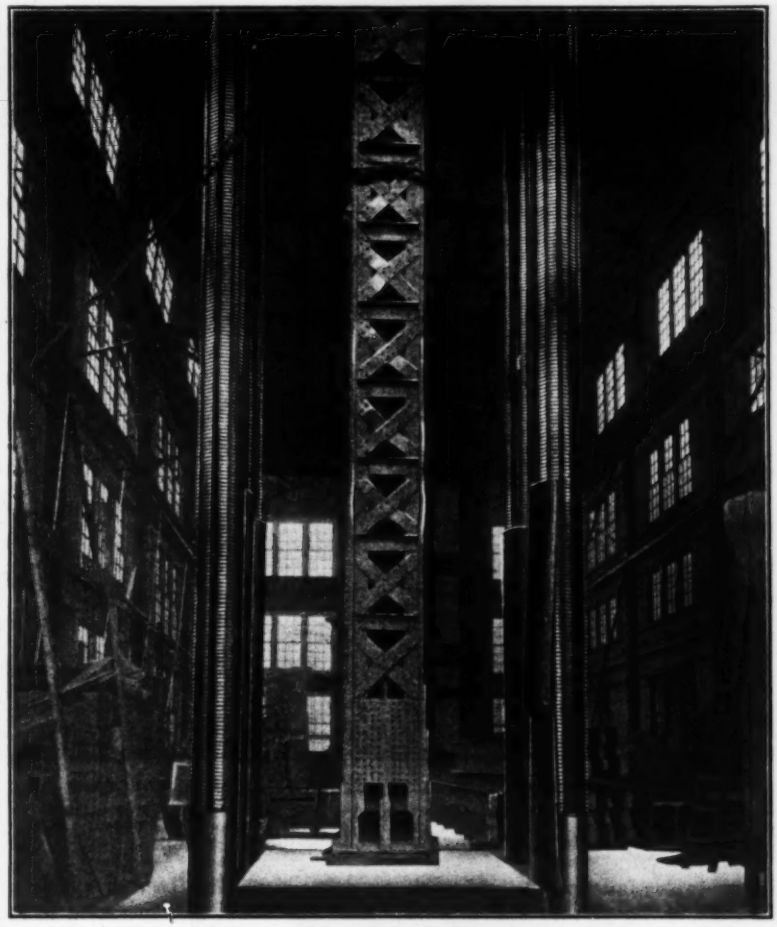
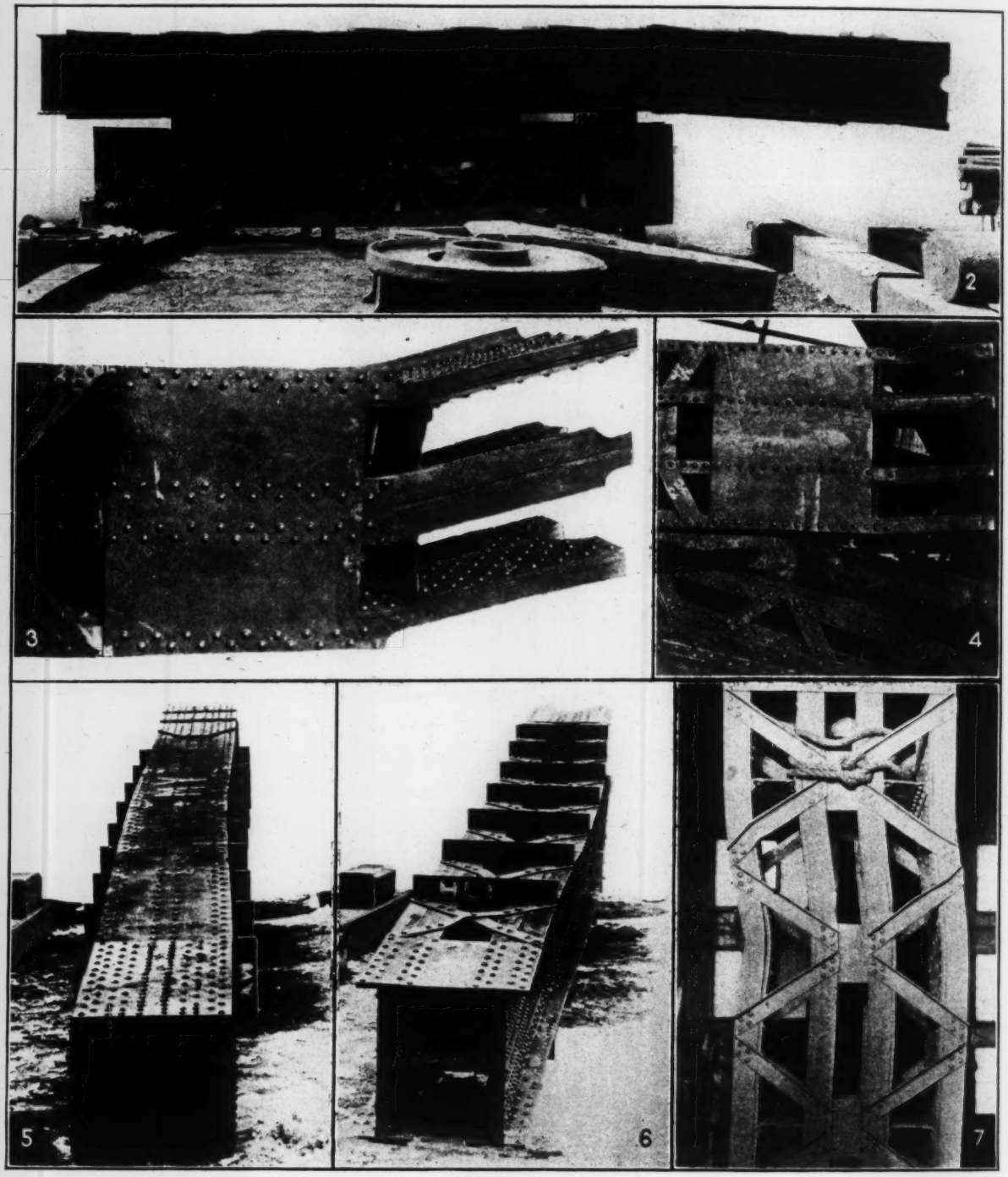


FIG. 1. THE BUREAU OF STANDARDS 5,000-TON TESTING MACHINE AT PITTSBURGH
A high-carbon steel column under test. Restrained lattice. Buckling of diagonals has begun

The test objects were half-size models of compression members of three very large bridges—two now under construction, one recently completed. None of these models were made of ordinary "bridge" steel; they were built of nickel steel, Mayari steel, chrome steel, silicon steel and high-carbon steel. The chemical compositions are tabulated alongside the test results, Table 2. The columns ranged in length from 16 ft. to 24½ ft., in cross-section from 42 to 119 sq.in., and in slenderness-ratio from 15 to 44. Important details of construction differed in the several columns. Some had pin jaws at the ends, involving the usual absence of lateral bracing or staying of the thin end sections. Some columns had simple crossed lattice, while others, particularly those of the Metropolis Bridge, had transverse struts between successive sets of lattice bars. The columns were tested in vertical position. Deformation was measured by 80-in. extensometers clamped to the four corners at midlength. Numerous measurements were taken also on 8- and 16-in. gage lengths, on shaft, lattice bars, pin plates, diaphragms, etc.

Supplementary tests were made on small specimens, to obtain information on the qualities of the material and to study the effects of additional strain from fabrication.



FIGS. 2 TO 7. DIFFERENT APPEARANCES OF FAILURE IN THE LARGE TEST COLUMNS

Fig. 2—Two typical body failures; local buckling of plates and angles with distortion of lattice. Figs 3 and 4. Buckling of pin jaws at end of laterally supported part. Figs. 5 and 6. Integral bending of column fitted with transverse stay members in the lattice. Contrast with Fig. 7, showing local buckling

Of the eighteen columns twelve failed as integral members, by bending in the body, as though they were of solid cross-section. The four columns with pin jaws failed by bending of the jaws. Two columns failed in the plane of greatest theoretical strength, distorting in the body.

No influence of column length was discernible in the results, as might be expected from the low values of slenderness-ratio and the small range under test. The bending at failure in the views is exaggerated; the load was increased to get enough distortion for a good picture.

Deflections first became visible at loads close to the maximum load, and increased to an inch or two as the loading progressed.

The average of the loads at failure is practically identical with the mean yield-point of the component steels (only ½% deviation). More usefully stated: The curve of ultimate strength is practically confined to the zone between the curves of high and low extremes of the tensile yield-points for the specimens (each column, being composite, contains material of different yield-points.

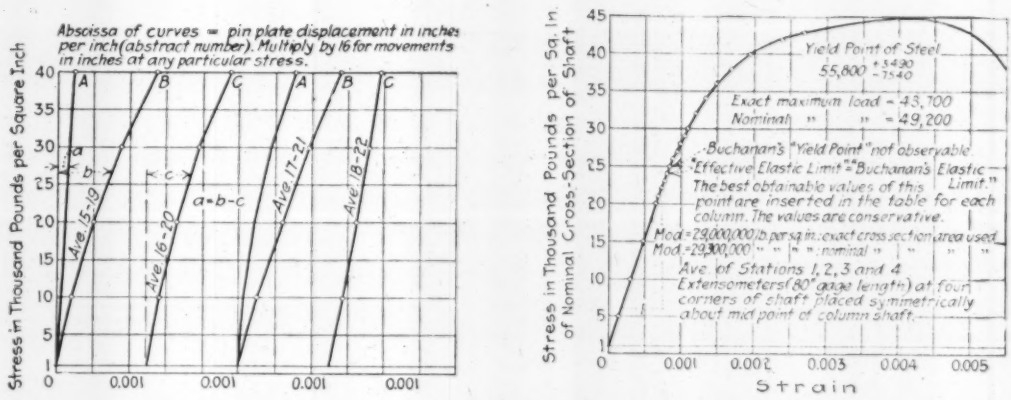

Abscissa of curves = pin plate displacement in inches per inch (abstract number). Multiply by 16 for movements in inches at any particular stress.
Stress in Thousand Pounds per Square Inch
A
B
C
a
b
c
Ave. 15-19
Ave. 16-20
a-b-c
Ave. 17-21
Ave. 18-22
0.001
Yield Point of Steel 55,800 +5490 -7540
Exact maximum load = 43,700
Nominal " " = 49,200
Buchanan's "Yield Point" not observable
"Effective Elastic Limit" = Buchanan's "Elastic Limit."
The best obtainable values of this point are inserted in the table for each column. The values are conservative.
Mod = 29,000,000 lb. per sq. in. exact cross section area used
Mod = 29,000,000 " " " " nominal " " " " "
Ave. of Stations 1, 2, 3 and 4 Extensometers (80" gage length) at four corners of shaft placed symmetrically about mid point of column shaft.
Stress in Thousand Pounds per Sq. In. of Nominal Cross-Section of Shaft
Strain
0.002
0.003
0.004
0.005

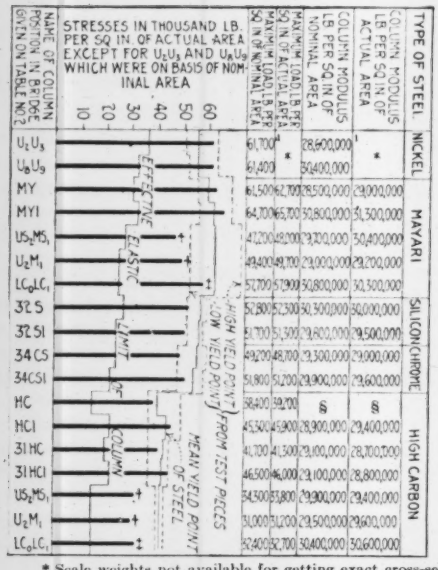
FIG. 8. TYPICAL CURVES OF TEST RESULTS

A typical load-deformation curve is given in Fig. 8. This shows a very straight lower portion and a gradually curving upper portion. The junction of the two portions is the elastic limit of the column. For the twelve columns that failed by integral bending the elastic limit averaged 48½% of the maximum load. The authors of the report believe, however, that the elastic limit was unfavorably influenced by initial strains from riveting and by

TABLE 1. DIMENSIONS AND MAKE-UP OF THE TEST COLUMNS

Bridge	Name of Column	Steel	Manuf.	"L" Length Pin. Cen. to End	Ends: Low. Pin, In.	Ends: Upp.	Cross Section of Shaft	Material of Shaft	Material at Pin End	Bracing of Shaft: Lattice, In.	Bracing of Shaft: Transv. Diaphgms, In.	Bracing of Shaft: Battens at Pin, In.	Area Sq. In.: Nominal	Area Sq. In.: Actual	Radius of Gyration R: ⊥ to Pin	Radius of Gyration R: ∥ to Pin	Slenderness Ratio = L ÷ R: ⊥ to Pin	Slenderness Ratio = L ÷ R: ∥ to Pin
Municipal, St. Louis	U_2U_3	Ni	American Bridge Co.	15 ft.–8½ in. End Top Chord	7½	Square	21¾" × 24½"	2 OUT. WEBS, 24" × ¾" 4 OUT. L's, 4"×4" × 7/16" 1 CENT. WEB, 24" × 11/16" 4 CENT. L's, 4"×4" × 5/16" 2 DIA. PLS. 9 1/16" × 5/16" 8 DIA. L's, 2"×2" × 1/4"	IN ORDER FROM OUTSIDE FOR U_2U_3 ONLY 6 PLS. 7/16" THICK 2 PLS. ½" THICK 2 PLS. 7/16" THICK 2 WEBS, ¾" THICK OUT. 2 PLS. ½" THICK 8 PLS. ⅜" THICK 1 CEN. WEB, 1/16" THICK	2½ Chan. (¼ web)	3 ⅛ thick	¼ pls.	88.8	no data	7.55	8.57	25.0	33.6
	U_0U_0	Ni		24 ft.–0 in. Center Top Chord	Square Pin not used			2 OUT. WEBS, 24" × 9/16" 2 SIDE PLS., 16" × 7/16" 4 OUT. L's, 4"×4" × 7/16" 1 CEN. WEB, 24" × 1/16" 4 CEN. L's, 4"×4" × 5/16" 2 DIA. PLS. 9 1/16" × 5/16" 8 DIA. L's, 2"×2" × 1/4"			3 ⅛ thick	¼ pls.	110.3	no data	7.78	8.97	37.0	32.1
Metropolis	HC	C	Penna. Steel Co.	All 24 ft.–4 in. Upper End Post	7	Square	19½" × 22½"	2 SIDE PLS. 22½" × 1" 4 L's, 4"×4" × ½" 1 DIA. PL. 17¼" × 7/16" 4 DIA. L's, 3"×3" × ⅜"		Transv. 4x3x⅜ in. angles diag. 4x¾ bars	3 ½ thick	¼ pls.	76.14	74.58	6.73	8.46 for all	43.5	34.5
	MY	My											76.14	74.90				
	31HC	C	American Bridge Co.										75.99	76.94				
	32S	Si											75.99	76.94				
	34CS	Cr											75.99	76.94				
	HC1	C	Penna. Steel Co.	All 24 ft.–4 in. Upper End Post	7	Square	17¼" × 22½"	2 SIDE PLS. 22½" × 11/16" 8 L's, 4" × 4" × ½" 1 DIA. PL. 15 3/4" × 7/16" 4 L's, 3" × 3" × ⅜"		Transv. 4x3x⅜ angles diag. 3x¾ bars	3 ½ thick	¼ pls.	76.46	75.35	7.60	8.05 for all	38.5	36.4
	MY1	My											76.46	75.28				
	31HC1	C	American Bridge Co.										76.26	77.17				
	32S1	Si											76.26	77.17				
	34CS1	Cr											76.26	77.17				
Memphis	US_2MS_1	C	Penna. Steel Co.	15 ft.–10 in. Upper End Post	7	Square	5¼" 12" 5¼" × 18"	1 COV. PL. 27" × 1/4" 4 WEB PLS. 18" × 7/16" 8 L's, 3" × 2" × ⅜" 4 FLATS, 2" × ⅜"	EACH COL. RIB 1 PL. ½" OUTSIDE 1 PL. ⅜" INSIDE 1 WEB, 7/16"	Transv. tie pls. ¼, see cuts. Diag. bar 3x¼	4 ½ thick	⅛ pl.	55.17	55.97	6.61	9.05	28.7	21.0
	US_2MS_1	MY		15 ft.–10 in. Upper End Post			27" × 18"	1 COV. PL. 27" × 7/16" 2 WEB PLS. 18" × 1/2" 6 L's, 3" × 3" × ⅜"	EACH COL. RIB 2 PLS. 9/16" OUT. 2 PLS. ⅜" IN. 1 WEB PL. ½"	Diags. 3x1¾x¼ angles	None	¼ pl.	42.47	41.66	6.87	9.78	27.7	19.4
	U_2–M_1	C		23 ft.–6 in. Upper End Post	8¼		5¼" 11" 7/16" × 21¾"	1 COV. PL. 25" × ¼" 4 WEB PLS. 21" × 11/16" 8 L's, 4" × 3" × ½" 4 FLATS, 3" × ½"	EACH COL. RIB 1 PL. ¼" OUTSIDE 1 PL. ½" INSIDE 1 PL. ⅜" INSIDE 1 WEB, 11/16"	Diags. 3x1¼x¼ angles	6 ½ thick	⅛ pl.	96.01	95.77	7.80	9.25	36.1	30.5
	U_2–M_1	My		23 ft.–6 in. Upper End Post			10½" × 21¾"	1 COV. PL. 27" × ¼" 2 OUT. WEBS, 21" × 17/32" 4 OUT. L's, 4" × 3" × ½" 2 FLATS, 3" × ⅜" 1 CEN. WEB, 21" × 19/32" 4 CEN. L's, 4" × 3" × ⅜" 1 FLAT, 6 1/2" × ⅜"	OUTSIDE RIBS (2) 2 PLS. ½" 2 PLS. 5/8" 2 PLS. 9/16" 2 PLS. ⅜" 2 PLS. 9/16" (CEN. RIB (1))	Diags. 3x1¼x¼ angles	5 ½ thick	⅛ pl.	66.55	66.02	8.17	8.47	34.5	33.8
	LC_0-LC_1	C		15 ft.–6 in., bot. chord at pier			9¾" 13½" 9¾" × 21"	4 WEBS, 21" × ¾" 4 SIDE PLS. 13" × ½" 8 L's, 4" × 4" × ½"	EACH COL. RIB 3 PLS. ⅜" 1 WEB, ¾" 1 SIDE PL. ½"	Tie pls. ¼ diag. 3x1¼x¼ ∠s	4 ½ thick	⅛ pls.	119.00	118.18	6.74	12.14	27.6	15.3
	LC_0-LC_1	My		15 ft., 6 in., bot. chord at pier			9¾" 13½" 9¾" × 19¼"	4 WEBS, 19" × 19/32" 8 L's, 4" × 4" × ½"	EACH COL. RIB 3 PLS. ⅜" 1 PL. ½" 1 WEB PL. 19/32"	Tie pls. ¼ diag. 3x1¼x¼ angles	4 ½ thick	⅛ pls.	75.12	76.47	6.87	11.59	27.1	16.0

TABLE 2. SUMMARY OF RESULTS OF COLUMN TESTS



Name of Column	Position in Bridge, Given on Table No. 2	Maximum Load, Lb. per Sq. In. of Nominal Area	Maximum Load, Lb. per Sq. In. of Actual Area	Column Modulus, Lb. per Sq. In. of Nominal Area	Column Modulus, Lb. per Sq. In. of Actual Area	Type of Steel
U_2U_3	Municipal Bridge, St. Louis	61,700	*	28,600,000	*	Nickel
U_6U_9	Municipal Bridge, St. Louis	61,400	*	30,400,000	*	Nickel
MY	Metropolis Bridge	61,500	62,700	28,500,000	29,000,000	Mayari
MY1	Metropolis Bridge	64,700	65,700	30,800,000	31,300,000	Mayari
US_2MS_1	Memphis Bridge	47,200	48,200	23,200,000	30,400,000	Mayari
U_2M_1	Memphis Bridge	49,400	49,700	23,000,000	29,200,000	Mayari
LC_0LC_1	Memphis Bridge	57,700	57,900	30,800,000	30,300,000	Mayari
32 S	Metropolis Bridge	52,800	53,300	30,300,000	30,000,000	Silicon-chrome
32 S1	Metropolis Bridge	51,700	51,500	29,800,000	29,500,000	Silicon-chrome
34 CS	Metropolis Bridge	49,200	48,700	29,300,000	29,000,000	Silicon-chrome
34 CS1	Metropolis Bridge	50,800	51,200	29,900,000	29,600,000	Silicon-chrome
HC	Metropolis Bridge	38,400	39,700	§	§	High carbon
HC1	Metropolis Bridge	45,500	45,900	28,900,000	29,400,000	High carbon
31 HC	Metropolis Bridge	41,700	41,500	29,100,000	28,700,000	High carbon
31 HC1	Metropolis Bridge	46,500	46,000	29,100,000	28,800,000	High carbon
US_2MS_1	Memphis Bridge	54,500	53,800	29,900,000	29,400,000	High carbon
U_2M_1	Memphis Bridge	51,000	51,200	29,500,000	29,600,000	High carbon
LC_0LC_1	Memphis Bridge	52,400	52,700	30,400,000	30,600,000	High carbon

Name of Column	Position in Bridge, Given on Table No. 2	Chemical Tests: Elements, % Ni	C	P	Mn	S	Si	Cr	Cu	Physical Tests: Yield Point Mean	High	Low	Break Load Mean
U_2U_3	Municipal Bridge, St. Louis	3.51	0.29	0.014	0.59	0.031	...	...	...	55,500	63,100	50,500	93,300
U_6U_9	Municipal Bridge, St. Louis	3.59	0.32	0.015	0.59	0.033	...	...	...	55,200	63,100	52,040	93,060
HC	Metropolis Bridge	...	0.36	0.012	0.43	0.030	...	...	...	40,700	50,000	37,590	76,660
HC1	Metropolis Bridge	...	0.36	0.012	0.40	0.030	...	...	...	44,100	50,000	37,650	75,930
MY	Metropolis Bridge	1.12	0.34	0.010	0.68	0.037	0.10	0.53	0.10	59,900	68,380	55,890	100,610
MY1	Metropolis Bridge	1.12	0.34	0.010	0.68	0.037	0.10	0.53	0.10	62,110	68,380	56,080	101,170
31HC	Metropolis Bridge	...	0.28	0.017	0.64	0.031	...	...	...	42,900	45,440	37,280	73,400
31HC1	Metropolis Bridge	...	0.28	0.017	0.64	0.031	...	...	...	42,700	45,440	37,280	74,400
32S	Metropolis Bridge	...	0.35	0.017	0.83	0.039	0.38	...	...	57,200	58,840	51,940	87,700
32S1	Metropolis Bridge	...	0.35	0.017	0.83	0.039	0.38	...	...	56,500	57,480	51,940	88,300
34CS	Metropolis Bridge	...	0.31	0.017	0.49	0.028	0.13	0.66	...	55,800	59,290	48,260	82,700
34CS1	Metropolis Bridge	...	0.31	0.017	0.49	0.028	0.13	0.66	...	54,200	58,010	48,260	83,600
US_2MS_1 carbon	Memphis Bridge	...	0.23	0.012	0.49	0.027	...	...	...	41,740	43,700	40,250	65,420
US_2MS_1 Mayari	Memphis Bridge	...	0.33	0.016	0.64	0.027	...	0.37	...	62,430	72,160	52,000	99,800
U_2M_1 carbon	Memphis Bridge	...	0.23	0.012	0.49	0.029	...	...	...	41,740	43,700	40,250	65,420
U_2M_1 Mayari	Memphis Bridge	...	0.33	0.016	0.64	0.027	...	0.37	...	62,830	72,160	52,000	99,800
LC_0LC_1 Carbon	Memphis Bridge	...	0.23	0.012	0.49	0.029	...	...	...	41,740	43,700	40,250	65,420
LC_0LC_1 Mayari	Memphis Bridge	...	0.33	0.016	0.64	0.027	...	0.37	...	62,430	72,160	52,000	99,800

* Scale weights not available for getting exact cross-sectional areas of shafts.
† Columns failed by local bending at end, as in Fig. 2.
‡ Columns failed as indicated in Fig. 3. Remaining members developed the strength as a unit.
§ Excluded in modulus determinations on account of the few observations taken within limits of proportionality. The 15 columns not marked (*§) were used for modulus calculations. Each stress and strain reading was treated as an observation.

crimping or local buckling of the shapes under increasing load.

In the ten columns of the Metropolis Bridge, all identical as to length and cross-sectional area, five of the specimens were built as H-sections in which each rib was a built channel with web and two flange angles, while the other five had in addition outer flange angles (and somewhat thinner web materials to keep the total-cross-section the same). Of the former set, four failed below the yield-point of the metal, while of the second set only two failed below the yield point. In other words, the columns with I-ribs were stronger than those with channel-ribs.

Transverse Members in Lattice

The Metropolis Bridge columns all had transverse bars at the panel-points of the lattice, and the lattice was double. More uniform bending of the columns was obtained and all sharp bends in the ribs eliminated. All these members failed with the smooth-integral-deflection curve exhibited in Figs. 5 and 6.

All the pin-plate observations indicate slight movement of one pin plate over the other. The intensity of the stress just over the pin probably accounts for this slip.

The modulus of elasticity of fifteen columns out of the eighteen averaged 29,600,000 lb. per sq.in., with a probable error of 120,000.

The coefficient of lateral expansion of the steel was found to be approximately the same as that of plain steel, the range being from 0.291 to 0.327, with a mean of 0.30.

Chicago Station Has 45,000-Hp. Steam Turbine

A 45,000-hp. (30,000-kw.) steam-turbine generating unit has recently been put into service in the Northwest station of the Commonwealth Edison Co., of Chicago. The following details of this huge unit are taken from an article in *Power*, June 20, 1916:

This machine is known as Unit No. 3 and has high- and low-pressure turbine elements on one horizontal shaft with the generator rotor (coupled sections).

Steam is carried in a 20-in. header from a battery of five boilers; in the high-pressure chest there are 14 admission valves all under control of the governor. There are also four overload valves discharging into the fourth stage. In passing through the high-pressure element the steam is expanded to 7 lb. below atmospheric pressure; it then enters the center of the low-pressure element and

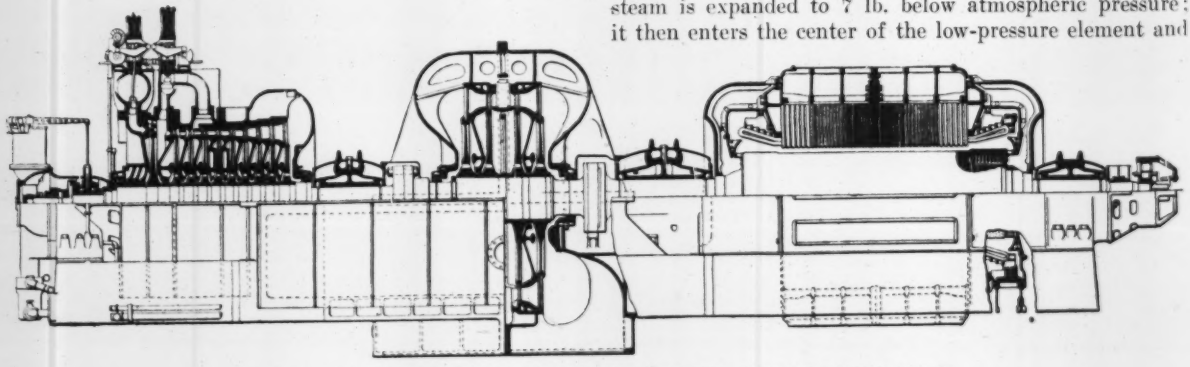


FIG. 1. PART SECTIONAL VIEW OF 30,000-KW. TURBO-GENERATOR

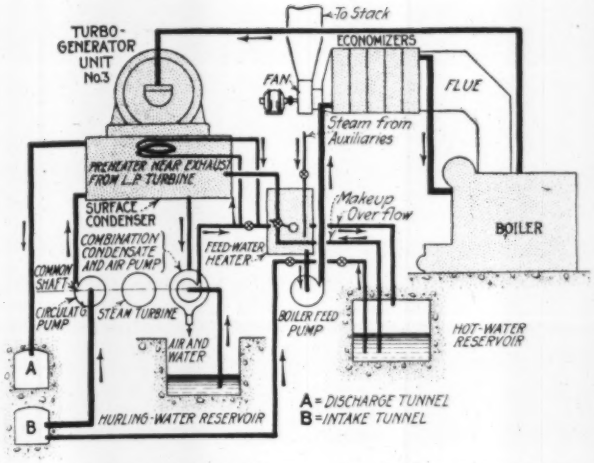


FIG. 2. DIAGRAMMATIC SKETCH OF STEAM AND WATER CIRCUITS

PRINCIPAL DATA ON EQUIPMENT OF UNIT NO. 3

Turbine

Maker	General Electric Co.
Type	Horizontal compound
Capacity, hp.	45,000
Number single stages, h.-p. element	10
Number of double stages, l.-p. element	2
Speed, r.p.m.	1,500

Condenser

Maker	Wheeler Condenser and Engineering Co.
Number of tubes	11,000
Size of tubes, in.	1
Surface in condenser, sq.ft.	50,000
Surface per kilowatt of generator rating, sq.ft.	1.67
Capacity, lb. of steam per hr.	360,000
Steam condensed per square foot of surface, lb.	7.2
Weight of condenser, empty, tons	176
Weight of cooling water in condenser, tons	66.5
Circulating-pump capacity, gal. per min.	52,000
Circulating-pump capacity, lb. per hr.	26,000,000
Cooling water per pound of steam, lb.	72
Condensate pump, gal. per min.	1,200

Generator

Maker	General Electric Co.
Capacity, rated	30,000
Voltage	9,000
Frequency, cycles	25
Speed, r.p.m.	1,500
Number field poles	2
Length complete unit, over-all ft.	59.5
Width, ft.	18.33
Floor area cover, sq.ft.	1,091
Area per kilowatt of generator rating, sq.ft.	0.036
Exciter voltage	220

Boilers

Maker	Babcock & Wilcox Co.
Type	Cross-drum, water-tube
Pressure, lb. per sq.in. gage	230
Superheat, deg. F.	200
Temperature of steam, deg. F.	600
Number of boilers in unit	5
Number of tubes per boiler	588
Diameter of tubes, in.	4
Length of tubes, ft.	18
Steam-making surface in boiler, sq.ft.	12,200
Stokers per boiler	2
Type of stoker	B. & W. chain-grate
Active area of two stokers, sq.ft.	273
Ratio grate area to boiler-heating surface	1 to 45
Per 1,000 sq.ft. of boiler-heating surface:	
Connected grate area, sq.ft.	22.3
Stack area, sq.ft.	4.17
Economizer surface, sq.ft.	538.2
Capacity of each boiler, lb. steam per hr.	85,000
Evaporation per sq.ft. of heating surface, lb.	7
Coal capacity of each boiler, lb. per hr.	12,600
Coal per square foot of grate, lb.	46
Size of steam main to turbine, in.	20
Boiler-feed pumps:	
Maker	Henry R. Worthington
Type	Turbine driven, three-stage, double-suction impeller
Capacity, lb. per hr.	450,000
Speed, r.p.m.	2,500
Water temperatures:	
In feed-water heater, deg. F.	100–120
Leaving economizer and entering boiler, deg. F.	270

Economizers

Maker	D. F. Sturtevant Co.
Type	High-pressure
Number of economizers	5
Number of tubes in each	456
Length of tubes, ft.	12
Heating surface in tubes	6,566
Maker	D. F. Sturtevant Co.
Type	Multivane
Capacity, cubic feet hot gases per min.	90,000
Horsepower of motor	100
Draft in boiler uptake, in. of water	2.4
Height of stack above boiler-room floor, ft.	250
Diameter of stack, inside, ft.	18

flows in opposite directions through double stages on either side and finally goes to the condensers. The steam and water circuits are shown diagrammatically in Fig. 2.

The shaft has a maximum diameter of 24 in. and is supported by four bearings. The latter are water cooled by pipes embedded in the babbitt; oil is supplied by forced circulation. The generator is a three-phase 25-cycle 9,000-volt machine rated for 30,000 kw. at 100% power factor (but it has carried 35,000 kw. with moderate temperature rise). A 220-volt exciter is mounted on the end of the main shaft.

The condenser is a two-pass surface type maintaining vacuum within ½ lb. of absolute. It will take 360,000 lb. of steam per hour, or 7.2 lb. per sq.ft. normally; the maximum steam-condensing capacity is 400,000 lb. The circulating water is drawn from an intake tunnel by a 48-in. double-suction tri-rotor pump driven by a 650-hp. noncondensing turbine. The normal hourly quantity of circulating water is 3,120,000 gal. On first entering the condenser the steam comes in contact with the preheating feed-water tubes. Air and condensate are taken from the condenser through a single outlet by a turbo air pump on the shaft with the circulating pump. In a separating chamber the air turns off into a separate compartment and is discharged by separate impellers. From Fig. 2 it is seen that the condensate passes from the heater to two turbine-driven boiler-feed pumps, thence to the economizers above the boiler and to the boilers. The accompanying table shows the principal data of turbines, condenser, generator and boilers for Unit No. 3.

Inaccurate Estimate Vitiates Time-Penalty Clause

The firm of Winston & Co., contractors, built a dam and reservoir for the City of Pittsfield, Mass. In the preliminary estimate of quantities placed in the sheet sent out for the information of bidders the quantity of earth excavation in stripping was estimated at 37,000 cu.yd. In actual construction the stripping amounted to 95,877 cu.yd. This increase developed gradually during the working season and could not be estimated in advance, as the engineer designated from time to time the depth to which the stripping was to be carried.

The contractors failed to complete the work on the specified date, and the city deducted liquidated damages of $75 per day for the time in which the contractors were delinquent. The latter brought suit for this amount, holding that the great discrepancy between the estimated quantity and the actual quantity of earth and other materials was the cause of the delay and that they should be relieved from the time requirement, which was based on the quantities in the original estimate. The case was tried before the United States District Court in Boston, Mass., Judge Morton presiding. The judge in his decision said:

> We have here a contract to do work the extent of which neither party knew when the work was begun. They had a rough-and-ready estimate which was nowhere near the facts. If the defendant chose to make that sort of indefinite contract, I do not think that he can hold the plaintiff to the penalty and liquidated damages if the work was not done on time. If the defendant wanted to hold onto that, I think it was the defendant's place to familiarize himself with the extent of the work in hand and give directions.

It should be mentioned that this was a preliminary decision, in which the case was placed in line for a jury trial of the question at issue.

Anchor-Bolt Holes Drilled by Gasoline-Air Outfit

By H. L. Hicks*

The New York Edison Co. has under construction an electric transmission line from Elmsford to Croton Lake, N. Y., a distance of 18 mi. For the greater part of its length it follows the line of the Catskill Aqueduct, and this eliminated the work of clearing the right-of-way.

The latticed steel towers are to be spaced at 400-ft. intervals and will be 60 ft. in height, except at angles and curves in the line where 70-ft. towers will be erected. The foundation work has been in progress for the past six months. The towers are to be bolted to 6-ft. steel anchor bolts, or "stubs," embedded in reinforced concrete. These stubs are 2 in. in diameter and are without heads, dependence being placed upon the holding power of

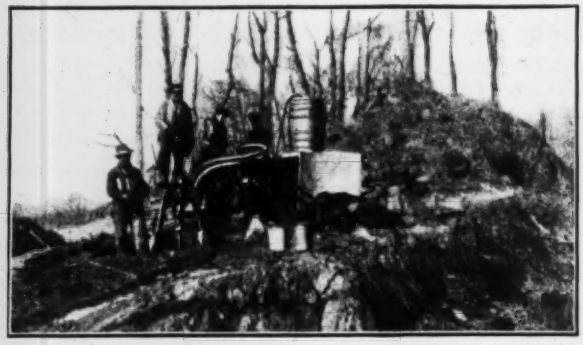


FIG. 1. SHOWING ROUGH GROUND OVER WHICH DRILL HAD TO BE TRANSPORTED

the corrugated surface. The right-angle flanges to which the towers will be bolted are 3 in. wide by 12 in. long and are drilled for six ¾-in. bolts.

In laying out the line it was found that at 12% of the locations rock would be encountered. This necessitated setting many of the stubs in solid rock and drilling large-diameter holes up to 5 ft. in depth.

The large amount of hole drilling made it advisable to employ machine drills, and the Ingersoll-Rand "gasoline-air" drill was selected for the work.

This is the first recorded installation of this type of drill in this section of the country. The gasoline-air drill (Fig. 1) consists of a tripod-mounted piston drill operated by a pulsator similar to the Temple-Ingersoll electric-air outfit. The pulsator in this case is driven by a 6-hp. single-cylinder gasoline motor which with fuel tank and pulsator is mounted on a small wheeled truck. The drill is reciprocated by pulsations of air acting in a closed circuit on alternate sides of the drill piston.

The pulsator unit weighs about 600 lb., the comparative lightness and compactness being a great advantage under the conditions.

For convenience in handling on soft ground the truck mounting the engine and pulsator was fastened to skids (Fig. 2) and a removable wooden housing provided as a protection from the weather. The entire outfit was transported by wagon.

Bedrock was uncovered and the drill set up in the excavation. Particular care in setting up was necessary to keep the drill perpendicular and the four holes accurately spaced. The rock was very seamy. If the hole started

*With Ingersoll-Rand Co., 11 Broadway, New York City.

FIG. 2. ENGINE AND PULSATOR ON SKIDS FOR EASIER MOVING

to run off, pieces of iron were wedged around the drill steel to correct this.

Collapsible steel forms were set up, the stubs were grouted in the drill holes and the concrete then poured. The anchoring stubs were spaced 2½ ft. each way, being located by a templet and so held in place until the concrete had set (Fig. 3).

The circular concrete foundation is 3 ft. 10 in. in diameter with a hollow core 2½ ft. in diameter and projects 6 in. above the ground. This center space is subsequently filled with spoil. Where no rock is encountered, the concrete is extended to a full depth of 5 ft.

Observations of the performance of the gasoline-air drill in drilling a group of four holes showed a total of 21 ft. of drill hole in 4 hr., including the time of changing set-up and squaring the layout of holes. These ob-

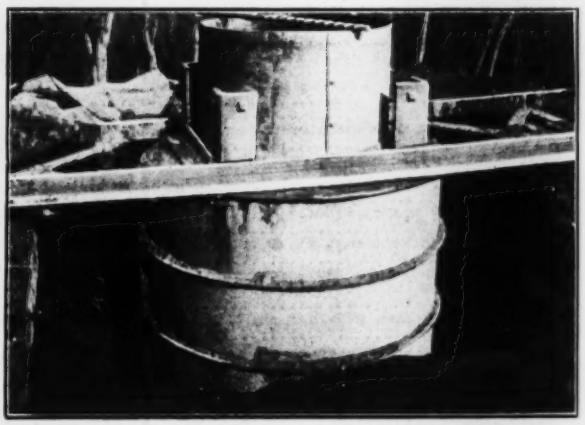


FIG. 3. FOUNDATION FORMS AND TEMPLET IN PLACE FOR POURING

servations were made on a day when the temperature ranged from 10° at 8 a.m. to 24° at noon. The fuel consumption averaged 1 gal. of gasoline and ⅕ pt. of oil per 10 ft. of drill hole. Crossbits were used, with drill holes bottoming at 2¾ in. The labor expense consisted of a drill runner at $2.50, a helper at $2.00 and an engine operator at $2.50 per day.

The drill is operated intermittently, and the drill crew is employed part of the time on other work. It is therefore rather difficult to arrive at any accurate estimate of the operating cost per foot of drill hole.

The erection of the steel towers is progressing, and the line is being rapidly pushed to completion.

Precise-Level Survey of the City of Portland, Ore.

By W. P. Hardesty*

SYNOPSIS—With city area of over 65 sq.mi. and hilly country—over 1,000 ft. difference of elevation—Portland lacked benchmarks. In a comprehensively planned survey over 2,000 bronze tablets and iron-pipe benchmarks were set, their elevations determined by precise levels and a full list published. The cost of the work was very low—net, about $4 per benchmark.

Portland had never had a comprehensive system of levels over any part of the city. The survey to be described was undertaken because of the inferior system of benchmarks, both as regards permanency and accuracy, that already existed. As there was considerable variation in different portions of the city and as there were but few really permanent marks, the survey included the accurate determination of points in widely scattered districts and also the establishment of what might be called "working benchmarks." The main result desired was the provision of thoroughly good, permanent, definite, easily located and accessible benchmarks, so distributed that one could be picked up in any locality where either preliminary surveys or improvement work was under way or was likely to be done.

The project was authorized by the city council and an appropriation made under the head of "Permanent Works." The survey to be made was officially named the "Bench-Level Survey." An outline of the system, methods and working rules, together with plans of standard benchmarks to be manufactured, was developed by the writer and approved by Robert G. Dieck, Commissioner of Public Works.

*Chief, Bureau of Surveys and Drafting, Department of Public Works, Portland, Ore.

Equipment and Methods of Survey

Equipment—Two precise levels were made to order for the work by Leupold & Voelpel, Portland. These are not Y-levels. The cost of each complete, with metallic mirror

FIG. 1. TABLET BENCHMARK, CLASS A, IN CONCRETE WALL OF GARAGE

FIG. 2. IRON-PIPE BENCHMARK, SIZE 1, WITH INVERTING PRECISE LEVEL ALONGSIDE

for reflecting level-vial scale, was $168. The instruments were made according to rigid specifications. One is erecting (18¼-in. length) and the other inverting (17½-in. length). Tripod heads with four leveling screws were used with these instruments.

There were also purchased two 10-ft. precise-level rods figured directly and two others with the figures inverted. These rods were in one piece, with a circular level mounted on the rod and a target reading by vernier to thousandths. The cost of the four rods delivered in Portland (manufactured in Seattle) was $75. The cost of smaller tools and items of equipment brought the total cost of the field outfit up to $427.50.

Ordinarily only one level, the inverting one, has been used, and all the work has been done by one party, as it was not thought justifiable to employ two crews on the survey, although the equipment had been purchased for two.

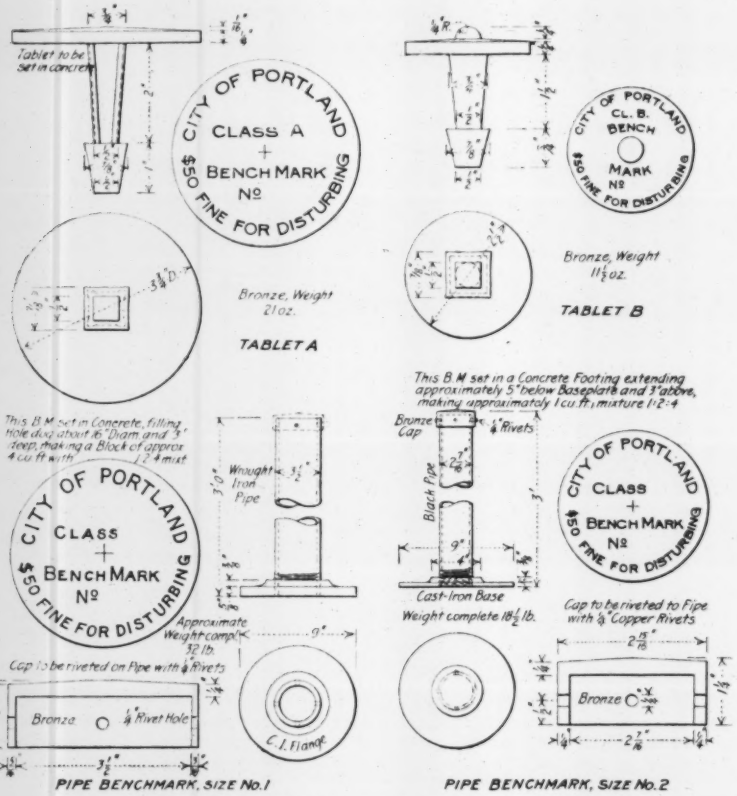


FIG. 3. DIFFERENT TYPES OF BENCHMARKS IN PORTLAND PRECISE-LEVEL SURVEY

Organization of Party—The party consisted of the levelman and three rodmen. The rodwork was done by two rodmen in the customary manner for precise-level work, while the third rodman carried the instrument box and smaller pieces of equipment and stayed with the instrumentman and kept a full set of peg levels as they were read.

In addition, each rodman kept a record of all the sights taken on his rod, so that the combination of the two afforded an additional full set of levels. The length of each level sight was also measured with the stadia wires of the instrument and recorded, and the sums of backsights and foresights were occasionally equalized.

Limits of Error—Nearly the same standards were adopted for the survey as are used on the United States Coast and Geodetic Survey and on the precise levels of the United States Geological Survey, except that a much greater allowance of error was considered permissible on lines with a large range in elevation than in comparatively level country. The following limits were adopted: On lines with range of elevation not exceeding 100 ft. per mi., $0.025\sqrt{\text{miles run}}$; with range of 100 to 200 ft. per mi., $0.035\sqrt{\text{miles run}}$; 200 to 400 ft., $0.045\sqrt{\text{miles run}}$; over 400 ft. per mi., $0.055\sqrt{\text{miles run}}$. "Range of elevation" means the total rise or fall, or the sum of the two where there were both. The work, as a rule, came far within the foregoing limits of error, and only one or two lines had to be re-run. The datum plane adopted was that of the United States Geological Survey, a shift of about 21 ft. from the old datum.

Before beginning the work the city was subdivided into 30 nets, varying in size and shape. The boundary of each net was first run in the forward direction and then the levels run in the reverse direction, so as to have a complete check on everything. After this were run the subdivision lines, which were termed crosscuts. Benchmarks of all kinds were established on the crosscuts as well as on the boundaries, or routes. The nets were platted on a map drawn to a 1,250-ft. scale, and the outlines of each one were plainly marked.

There were two general types of manufactured benchmarks used—(a) bronze tablets and (b) sections of wrought-iron pipe surmounted by a bronze cap. Of the tablets (see Fig. 3), Class *A*, the larger, is intended for the more permanent locations, such as in or on concrete walls, on bridge abutments, etc. Class *B* is placed in the more available locations, generally in concrete curbs, sometimes in concrete walks or on walls. A benchmark placed in a wall (as Class *A* more commonly was) is not nearly so convenient to use, because it requires a knife or carpenters' chisel to support the level rod.

The iron-pipe benchmarks were of two sizes, the design being nearly the same. Size 1 was first used, but it was decided to be unnecessarily large and expensive, so that after the first order of 50 was used up a smaller one (size 2) was adopted. Either one may be set in concrete (either in a footing or to nearly the full height) or may be planted in earth only.

These four kinds of benchmarks were designed after careful study of those used by the United States Geological Survey and of those adopted by Cincinnati and Pittsburgh, and they are believed to be improvements on all of these.

All of size 1 pipe benchmarks were set in concrete reaching to within about 6 in. of the top, the block of concrete thus containing over 4 cu.ft. So substantial are

they that one in firm earth could well serve as a mooring post for a steamboat.

The first cost of the benchmarks (in lots of 30 to 100) was as follows:

	Each
Tablets, class A	$0.60
Tablets, class B	.30
Pipe, size 1	2.20
Pipe, size 2	1.75

In the case of the latter it was found that the total cost of size 1 in place (including the benchmark itself, labor of setting, team hire in transportation and cost of cement, sand and gravel) ran about $5 each, while that of size 2 ran about $3.

During the latter part of the survey a number of the smaller size were set without concrete, and these were called Class *B*. These, firmly planted in a hole and with the earth well tamped to above the ground, are believed to be more substantial than those of the United States Geological Survey (all similarly set), as the latter merely have the bottom of the iron pipe split to widen the base, while the Portland benchmark has a cast-iron base of 9-in. diameter screwed onto the bottom of the pipe. This gives a broad, flat base and also renders the tamped backfilling more effective.

All the pipe benchmarks were set with the top 6 to 10 in. above the ground.

Placing the Benchmarks—One of the first steps in the work was starting an instrumentman on a reconnoissance of the city (taking one net at a time) to select the best places for the benchmarks. His instructions covered the selection of suitable objects on which the tablets were to be set and of suitable spots for the pipes.

The matter of proper location and distribution of the sites was a vital one. The locator carried detail maps for each area in which he worked, and these showed all the established street-grade points. The frequency of benchmarks was logically greater in a rolling or hilly section than in a nearly flat one, for the convenience of future use. The instrumentman marked the points selected, using red bicycle enamel for concrete surfaces and stakes for the pipe sites. These marks and one copy of the duplicate loose-leaf notes taken describing the location were for the guidance of the benchmark setter later on. The other copy of the loose-leaf notes was for the level party.

It may be remarked that the frequency of benchmarks was largely dependent on the objects available for the tablets. In the localities not built up, especially those with very few sidewalks, dependence was placed mainly on pipes, and the number of these was necessarily limited.

The benchmark setter was a mechanic skilled in cement work. With a set of tools (small drills, chisels, single hammer, etc.) he made the rounds, drilling holes in the concrete surfaces selected and setting the tablets in a 1:1 cement mortar. For setting the pipes a wagon with team and driver was added for hauling the benchmarks, cement, sand, gravel and tools. The benchmark setter was often several weeks behind the reconnoissance man and at times was several months ahead of the level party.

To secure proper objects for the tablets and proper sites for the pipes frequently required going onto private property, and the written consent of the owner to the placing of the benchmark was in each case obtained. Two forms of permit were used—one for corporations and the other for private individuals. In all considerably over 100 permits had to be obtained.

In addition to the manufactured benchmarks the elevations were taken on the existing street monuments wherever the latter could be reached from the lines of levels. The street monuments are considered as being of about the same order as the Class *B* manufactured benchmarks.

Advantage was taken of the opportunity in running the levels to establish a large number of additional benchmarks on available points and objects. These were more commonly points on the concrete curbs, generally either on ring bolts in the curb or on one end of an angle-iron curb-guard at the sidewalk corners. These benchmarks are called Class *C*. While not so permanent and definite and so certain of identification in the field as the others, they are fully as good as the general run of benchmarks on most engineering works. These benchmarks were determined with the same degree of accuracy as the others.

It will be observed that only the manufactured benchmarks could be numbered, as the design provided space for the serial number to be placed. The numbers were assigned in consecutive order as the benchmarks were determined, without any regard to locality. The numbers were stamped on with a steel die. This numbering of benchmarks is a marked advantage, both as regards certainty of identification and ease of reference in tabulations and on maps. On the benchmark map constructed from the survey, each numbered benchmark was located in position and the number entered alongside, while the street monuments were also noted. Different symbols and colors were used for the different classes and types.

Extent and Progress of the Work

Fieldwork by the level party began about the middle of March, 1914, and was finished at the end of July, 1915. The work was continuous except for an intermission of about 2½ mo. Over 500 mi. of level lines was run. As much of Portland's area is quite hilly, progress could not be so rapid as in a level locality. The extreme range in elevation of benchmarks set was 1,017 ft.

Area Covered—The total area within the Portland limits (including some water surface) is now 66.3 sq.mi. Of this area, 9.5 sq.mi. is taken by the recent annexation of Linnton, and the survey was not extended over Linnton. Practically all of the remainder of the city was covered, and several of the nets extended outside of the city. Over 6 sq.mi. outside the limits was thus covered, and an additional area of 2 sq.mi. (some of it in a different county) was traversed to the extent of running some main circuits.

Altogether about 65 sq.mi. gross area was covered by the survey. The frequency of benchmarks ranged from 13 to about 60 per square mile within the city. Outside the city limits some areas have only 4 to 5 benchmarks per square mile.

Tabulation of Benchmarks—On the survey there were established and accurately determined in elevation the following number of benchmarks: 104 *A* tablets, 973 *B* tablets, 179 *A* iron-pipe benchmarks, 17 *B* iron-pipe benchmarks, 148 street monuments and 585 *C* benchmarks, making a total of 2,006. In addition, 17 United States Geological Survey benchmarks or points were checked on and the differences noted. All these benchmarks were listed in the tabulation afterward made.

The arrangement of such an immense number of benchmarks in a list for ready reference was a formidable undertaking, especially so because of the great diversity

of the street systems and the irregularity because of topographical considerations existing in Portland. The benchmarks in any city are usually referred to by their street locations, though sometimes better designated by other features, such as railway lines, river or harbor lines, etc.

It was decided to divide Portland into eight areas or districts and to list the benchmarks in each one separately. A general alphabetical arrangement for the entire city did not promise well, because of the great number of street names and street numbers. An additional consideration was the fact that the list was to be published in loose-leaf form. As the work of many of the city surveyors is confined to certain sections of the city, the listing by districts permitted the separate use of just those leaves needed to cover the particular districts affected. The surveyor can thus place whatever leaves are necessary in his loose-leaf field binder with his regular survey notes, the size and punching of the regular survey leaves being the same as those of the benchmark publication.

The method used in listing was to compile a "consecutive" index of all the benchmarks in each district, supplemented by a short alphabetical list of those benchmarks where the street locations did not fall within the regular scheme. In addition, all of the nearly 1,300 numbered benchmarks were listed in a separate table in their numerical order. By this latter list, if the number of a benchmark be known, it can at once be turned to.

The benchmark tabulation was printed in booklet form in January, 1916. The booklet of 141 pages, of 4½x7¼-in. size, contains all the different lists, with full prefatory notes, explanations, table of contents and index. The issue consisted of 350 copies bound in flexible covers and also 350 sets of loose leaves (each set with loose-leaf covers for convenience of keeping).

Cost of Survey and Tabulation—Following are items composing the complete cost of the survey from the beginning of the preliminary work to the printing of the tabulation:

Salaries of field men	$5,076.85
Wages of benchmark setter	773.50
Salaries of office men in checking, adjusting, distributing errors of closure, indexing, compilation of lists of benchmarks, etc.	848.80
Hire of team and driver for hauling pipe benchmarks and materials for setting	145.25
Cement, sand and gravel for concrete for pipe benchmarks	54.39
Manufactured benchmarks of the four types (about $70 worth not yet used)	805.20
Equipment, including minor supplies for surveying	435.58
Street-car fare for employees	89.60
Miscellaneous items	33.54
Publication of the tabulation of benchmarks (350 bound copies and 350 loose-leaf copies)	245.00
Total	$8,507.71

If the value of the equipment (less some depreciation) and the value of the supplies—all available for future extensions—be allowed, the net cost of the survey reduces to not over $8,100. The *pro rata* cost for the 2,006 benchmarks established thus becomes about $4 each.

Attention is now called to the fact that a primary aim of this survey was to establish working benchmarks, as referred to at the beginning. In the precise-level surveys of other cities, such as those of New York and Chicago and to some extent those of Pittsburgh and Cincinnati, the primary object has been to carry precise levels out from an initial point so as substantially to cover the entire area of the city, but with the actual establishment of only occasional benchmarks. In these cities there was but little attempt made to put in working benchmarks, and the reference benchmarks established in the different sections or boroughs were depended on mainly as starting points. Secondary levels were naturally run from them by the regular city survey parties as occasion demanded, with the ordinary grade of leveling instruments. In Portland the aim has been to obtain a complete list of benchmarks for use by the city surveyors and others, the distribution being such that it will be seldom necessary to establish other intermediate benchmarks.

The covering of the city in this thorough manner was made all the more necessary by reason of the fact that nearly all the ordinary leveling in Portland has been done with transits, so that not a very high degree of accuracy is obtained. By having the permanent benchmarks at convenient intervals there will be no objection to continuing this practice, and work that would be needed in other cities in carrying out secondary sets of levels and the establishment of secondary benchmarks will be practically eliminated. For this reason the instrumental work in the future will be much lessened, and the cost of this bench-level survey could be very properly distributed over a number of years to come.

It will of course be necessary to extend the benchmark system with the growth of the city, both because of the expansion and annexation of contiguous territory and because of the building up of areas at present but thinly settled and in which but a limited number of benchmarks was established at the time of the survey by reason of so few available objects being at hand.

The project of covering the City of Portland with a comprehensive and precise level survey was initiated by Robert G. Dieck, Commissioner of the Department of Public Works, soon after his assumption of office in 1913. The execution of the survey and the resulting compilations have been under direct supervision of the writer.

Changes in New Specifications for Paving in St. Louis

Since the adoption of the new charter in 1914 the City of St. Louis has abandoned the practice of having standard specifications for each form of street improvement. The Designing Division, of which W. W. Horner is chief, now prepares individual specifications for each street; and, while the principal features are the same in all, the attempt has been made to work out carefully the special details applicable only to the particular improvement. For this reason it is impossible to print complete specifications. The division now has loose printed sheets for certain common clauses, and the additional sheets are typewritten for each case. Some of the special details, which would be difficult to describe in specifications, are shown with accompanying notes on the plans for the work. (Typical plans for street improvements in St. Louis were described in *Engineering News*, Sept. 2, 1915.)

The concrete base for the majority of the improvements is being made 5 in. in thickness, but the base has been increased to as much as 7 in. in particular cases. Some of the main points in the new specifications are as follows:

Wood-block pavement consists of 3½ in. yellow pine blocks treated with 16 lb. of heavy oil. The blocks are laid on a ¾-in. dry-mortar cushion with bituminous-filled joints. The filler generally specified is a high-grade

asphalt, but on one large contract special pitch filler is called for.

There have been no expansion troubles on any of the wood-block work, but on two pavements laid late last fall the blocks have bled rather badly during May and June of this year. Blocks laid this year and having apparently had a little longer final steaming do not as yet show any evidence of serious bleeding.

Of the very large yardage of vitrified-brick pavement in St. Louis, a considerable proportion is in bad condition because of the unsatisfactory sand bed that was used, and a number of contracts are now being let to re-cover these old brick pavements with an asphalt wearing surface.

Old Brick Pavements Surfaced with Asphalt

Specifications for resurfacing old brick with asphalt have been provided to meet three different conditions: First, where the old brick wearing surface is badly out of level on account of shifting of the sand cushion, the specifications provide for a rough readjustment of the brick. The pavement is then covered with binder in order to fill any remaining surface inequalities, after which a standard 1½-in. asphalt surface mixture is laid. Specifications provide for payment for the binder course by the ton and for the wearing surface by the square yard. They also provide for readjustment of curbing, for taking up old brick and for resetting old brick.

In the second case, on certain of the more important streets, it is proposed to provide new brick gutters. These gutters will be on a sharp slope in order to give a shoulder at their outer edge, against which the resurfacing material can be laid, and at the same time give an ample depth of curb face at their inner edge. In the third case, if a great portion of the old brick wearing surface is undisturbed and in good condition, it is proposed to readjust and carefully regrout all rough places, apply a thin paint coat of asphalt cement and a 1½-in. asphaltic wearing surface.

On account of the high cost of readjusting the brick in advance of the covering, as under the first condition noted, it appears doubtful whether this type of construction is warranted, because for a very small increase in cost the brick could be removed entirely, a small amount of concrete added and a standard sheet-asphalt pavement laid.

Brick Pavements for Streets and Alleys

Specifications for vitrified-brick pavement now provide for a wire-cut lug block, not repressed. On all street work the brick is laid on a ¾-in. bedding course of dry mortar of a 1:4 mix and is filled with a 1:2 mixture of portland-cement grout. For the alley pavements the brick are laid on a bedding course of limestone screenings.

During the past year three contracts have been let for brick-paved alleys, where a monolithic type of construction is provided for; that is, the brick and the mortar bedding course are to be laid on the fresh concrete and rolled with a hand roller. Prices on this work were approximately the same as for the first type mentioned, but none of the three alleys have as yet been constructed. It is doubtful if many alleys will be let on this specification, as the difficulty of carrying on this type of construction in a closely built-up alley would make the work unduly expensive.

One street having a 36-ft. roadway was paved by the monolithic method with very satisfactory results, but it was found that the wide roadway made the operations much more difficult than those on the usual narrow highway, so that the cost was materially greater than for the dry-mortar bed. No further attempt has been made to apply this method.

New Bituminous-Macadam Pavements

Following the decision of the Board of Public Service at the beginning of the fiscal year that no more waterbound telford macadam pavement would be laid, a new specification for telford was adopted, which provided for a 6-in. telford base, 3 in. of waterbound macadam and a 3-in. top course of bituminous macadam.

A high-grade asphalt has been specified for the binder in most of this work, but on one large contract refined tar was called for. On one contract also alternate bids will be taken for a base of 6-in. telford plus 3 in. waterbound macadam on the one hand and a 5-in. concrete base on the other.

More Concrete Curbing Being Used

Up to last year the curbing used had been almost entirely granite. Some limestone was tried several years ago, but it has given poor service. Nearly all the granite curbing has come from Georgia quarries and was formerly delivered for from 55c. to 58c. per lin.ft. on the cars. Since last year, when the specifications for curb were revised and a little better workmanship required, the prices have risen steadily until the quotations in St. Louis are now about 75c. per lin.ft. This is apparently due somewhat to the revision of the specification, to close inspection and largely to labor conditions at the quarries.

The division began last year to specify concrete curb in connection with bituminous-macadam work; and since the rise in price of granite, concrete curb is being specified on all except the most important streets.

Eliminating Fieldwork

The tendency is to make careful designs and preliminary estimates for each piece of work and to take bids in as much detail as possible in order that the contractors may have no occasion to estimate contingencies.

On account of the general increase in prices of materials and labor since these specifications were adopted it is difficult to ascertain to what extent the present form of plans, specifications and bidding is affecting the cost of work, but there is no doubt that it has tended toward simplification of the work in the field.

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Depreciation in Cost Accounting—The Federal Trade Commission is urging on every business man the absolute necessity of making proper provision for depreciation and doing it monthly or at least quarterly. I think I can safely say that no accounting system will receive the indorsement of the commission unless it does provide for the inclusion of this most important item of cost. Statistics show that the percentage of business men who do not provide for depreciation is very large, running over 50%, and this is one of the causes that have a great influence on the business death rate. Nearly every man is perfectly willing to include in his cost all items for which he pays out actual money, but he is inclined to overlook items, such as depreciation, that do not require a visible outlay. Many manufacturers and merchants do not charge any depreciation and give as a reason that they keep their plant and stock in first-class condition. This is one of the erroneous ideas in business today. Every machine, building and apparatus, like every man, has a certain period of life; and no matter how much care is taken of equipment, death is bound to come.—From an address by E. N. Hurley, Chairman, Federal Trade Commission.

Building the Rolling-Crest Dam Across Grand River

SYNOPSIS—The largest steel rolling-crest dam in this country has just been completed by the United States Reclamation Service over the Grand River near Grand Junction, Colo. In "Engineering News," July 6, 1916, F. Teichman, one of the engineers in the service, described the design of the dam. This article, prepared by one of the editors, describes the construction features.

Work on the Grand River dam was begun Aug. 27, 1913, but owing to the European War and unforeseen difficulties in arranging for the fabrication of the steel rollers and operating mechanism the construction operations were prolonged far beyond the original estimate.

Essentially the dam consists of a standard concrete masonry ogee weir, the crest of which is at El. 4,782, with an apron about 33 ft. long and 2 to 6 ft. thick with its top at El. 4,774, a little below the normal bed of the Grand River, and a front apron covered with puddle. On the weir is 420 ft. of rolling crest, 10 ft. 3 in. high, in six units of 70 ft. each. At the west end of the dam is a 60-ft. sluiceway controlled by a rolling crest of the same type, but 15 ft. 4 in. high, as the elevation of the sluiceway sill is 5 ft. lower than the crest of the ogee weir. The roller crests are of a type having its origin in Germany, and are built under patents owned by the Machinenfabrik Augsburg-Nurnberg A. G.

The right-hand (shore) wall of the sluiceway contains the canal intake regulator gates, nine in number, each 7x7 ft. The sill of the intake gates is at El. 4,785.25, or 8.25 ft. higher than the sill of the sluiceway and 6.25 ft. higher than the floor at the entrance to the sluiceway. The maximum height of water in the canal is 4,791.5 ft. for a discharge of 1,425 sec.-ft. The gates cost $6,079, the Hinman Hydraulic Manufacturing Co., Denver, being the contractor.

Ogee Weir and Piers

The dam substructure rests on gravel. Borings made 30 ft. in the bed of the stream showed the bedrock to be soft sandstone. The cross-section of the weir was made long to restrict the flow of the undercurrent and unusually heavy to resist sliding.

The diversion dam and canal headgates are one structure. The canal draws water at right-angles to the current so as to have the least interference from drift, and water is drawn from the upper strata so as to have a minimum of silt. The axis of the ogee weir is at right-angles to the river, except for a slight bend at the east end to meet the wall of the adjoining Orchard Mesa canal at right angles.

Construction of a dam at the site chosen was complicated by the presence of the tracks on the Denver & Rio Grande R.R. side and the canal of the Orchard Mesa Irrigation District. Both the railway tracks and canal bank had to be permanently raised and protected. The original intention was to supply the Orchard Mesa canal from this dam, but this arrangement could not be satisfactorily made with the Irrigation District.

The general scheme of construction was to build the sluiceway first and then, diverting the river through it, build the ogee weir and piers. A trestle was built across the river a short distance above the dam and filled with rock and earth to form a coffer-dam. The work was done under difficulties as the highest water on record occurred during this time. A cableway was erected over the center of the dam for handling materials.

All machines were electric driven. Power was obtained from the steam-generating plant at Cameo over a transmission line three miles long. Transmission was made at 16,500 volts a.c. and was stepped down by transformers in the substation to 2,200 volts a.c. and converted into 250 volts d.c. for all equipment. Sand and stone for the concrete was taken from a sandstone quarry on the east bank, opposite the dam. The over-burden as removed was used to tighten the coffer-dam and for the puddle on the upstream apron. The rock was handled by a stiff-leg derrick which dumped onto the feeding platform for a Gates gyratory crusher and Davis sand rolls. These discharged into the same bucket elevator running to a revolving screen mounted over hopperbottomed bins. Gates in the bottom of the bins emptied into measuring boxes for the mixer.

Negotiations for Fabrication of Rollers

In the latter part of 1913 it was considered that both time and money would be saved by having the steel rollers designed and fabricated by the patentees, the Machinenfabrik Augsburg-Nurnberg A. G. The German firm ac-

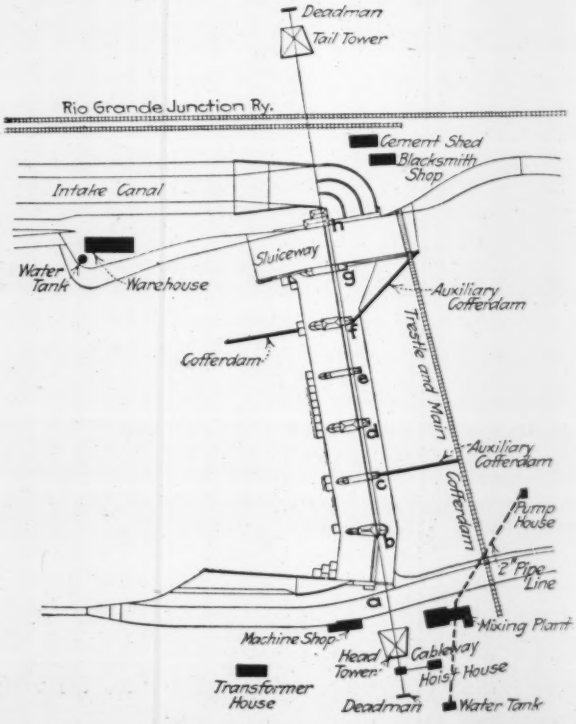
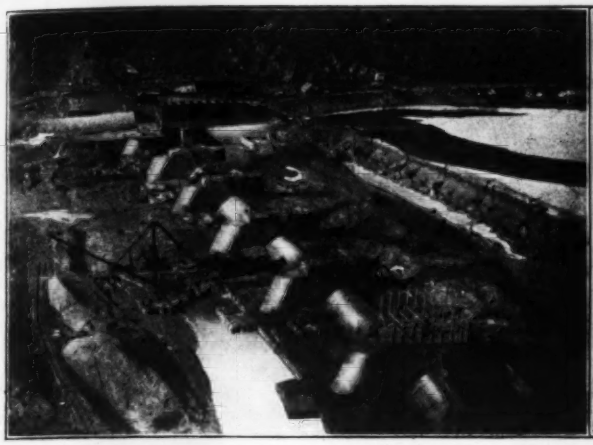


FIG. 1. GENERAL LAYOUT OF GRAND VALLEY DIVERSION DAM



FIGS. 2 AND 3. VIEWS OF GRAND VALLEY DIVERSION DAM DURING CONSTRUCTION

Fig. 2—Construction view, taken from the east side looking west toward sluiceway and canal headgates while dam site was still dry. Fig. 3—Same view as in Fig. 2; taken after rollers were up

cepted the contract in September, 1913. A set of plans was received at the Washington office of the Reclamation Service on Apr. 11, 1914, and submitted to the project office where they were reviewed and reported upon May 22, 1914. Fabrication of certain parts was begun in Germany.

Early in August, 1914, it became evident that because of the European War the German company would not be able to complete its contract and tentative plans were made for beginning the preparation of drawings and specifications for American manufacturers. Later the German company announced its inability to proceed with the work, and F. Teichman, Designing Engineer, Reclamation Service, proceeded at once to Washington and in the course of a few months entirely worked over the German designs to conform to American practice. C. H. Branscombe assisted in this work. Plans and specifications were completed and submitted to bidders Dec. 23, 1914. Owing to the fact that this is the first steel roller dam fabricated in this country the following list of bids and bidders is of interest:

Name of Bidder	Unit	Total
Engineers' estimate	3.5c.	$20,531.00
Riter-Conley Manufacturing Co., Pittsburgh, Penn.	2.55c.	14,958.30
Pennsylvania Bridge Co., Beaver Falls, Penn.	2.94c.	17,246.04
Stacy-Schmidt Manufacturing Co., York, Penn.	3.2c.	18,771.20
American Bridge Co., Baltimore, Md.	3.2c.	18,771.20
Independent Bridge Co., Pittsburgh, Penn.	3.46c.	20,296.36
St. Paul Foundry Co., St. Paul, Minn.	3.55c.	20,824.30
Minneapolis Steel and Machinery Co., Minneapolis, Minn.	3.72c.	21,821.52
Milwaukee Bridge Co., Milwaukee, Wis.	3.72c.	21,821.52
Chicago Bridge and Iron Works, Chicago, Ill.	3.75c.	21,997.50
Allis-Chalmers Co., Milwaukee, Wis.	3.8c.	22,290.80
Reummeli-Dawley Manufacturing Co., St. Louis, Mo.	3.9c.	22,877.40
Pittsburgh-Des Moines Steel Co., Pittsburgh, Penn.	3.97c.	23,289.02
Kansas City Structural Steel Co., Kansas City, Mo.	4.0c.	23,464.00
McMyler Interstate Co., Bedford, Ohio	4.76c.	27,922.16
Treadwell Construction Co., Midland, Penn.	5.375c.	31,529.75
Whitehead & Kales Iron Works, Detroit, Mich.	5.8c.	34,022.80
William Cramp & Sons, Philadelphia, Penn.	8.55c.	50,154.30

The contract was let to the low bidder, the Riter-Conley Manufacturing Co., of Pittsburgh, for $14,958.30. The erection and freight costs are not included in this or the other figures given. The erection was done by the Reclamation Service forces.

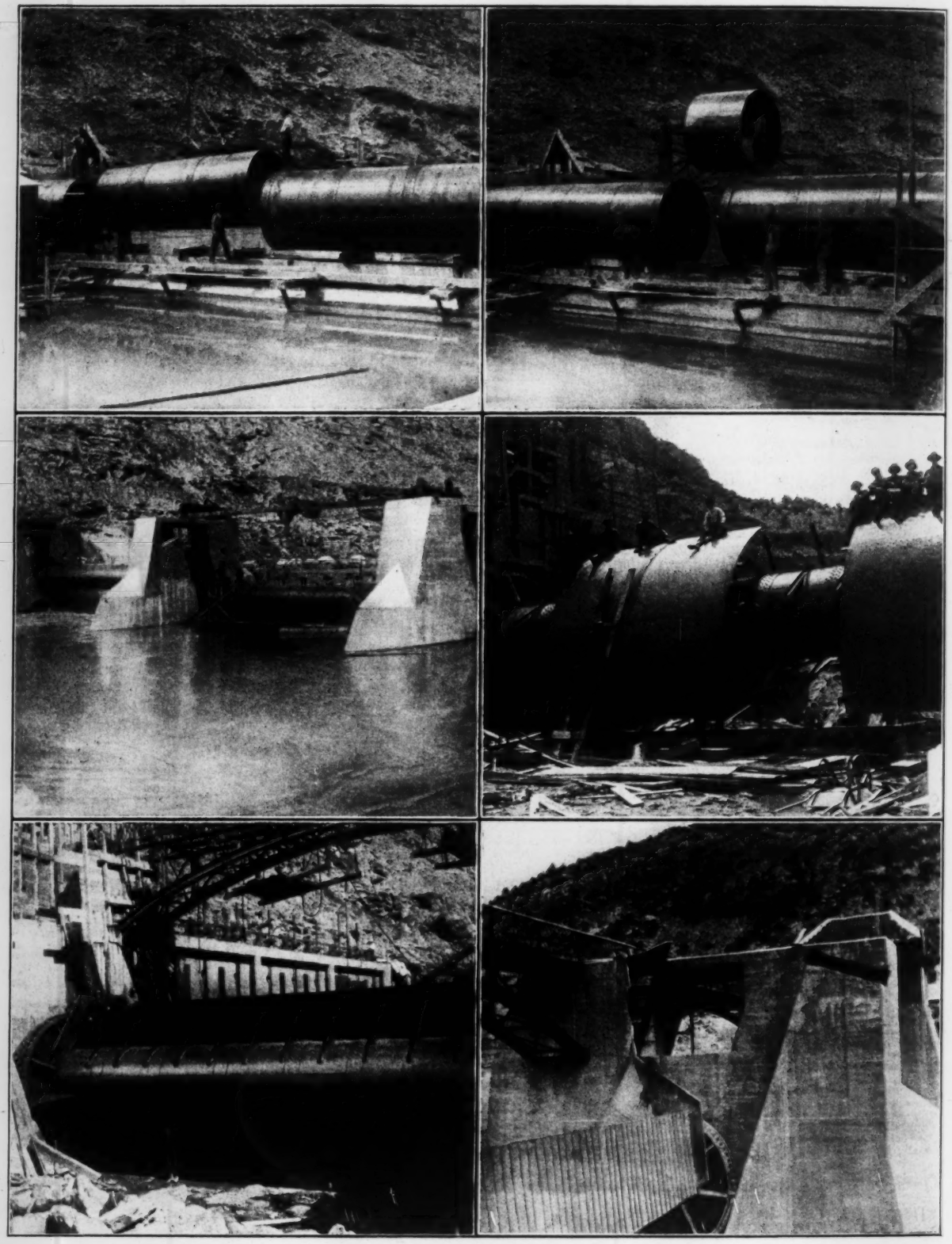
Having arranged for the manufacture of the rollers in this country it was still considered desirable to attempt arrangements with the German patentees for the manufacture of the hoists and chains. The hoist for the heaviest—the 60 ft. roller—was ordered, but orders for the hoists for the 70-ft. rollers were held pending bids from American firms. The bid of the Minneapolis Steel and Manufacturing Co., Minneapolis, Minn., was as low as the German bid and the contract for the six 70-ft. roller hoists was let to this firm for $825 each. The order for the 60-ft. roller hoist placed in Germany was then recalled and let to the Link-Belt Co., Chicago, Ill., for $1,600.

The only parts then left in the hands of the German manufacturers were the steel chains and sprockets, including shafts, but eventually these too had to be contracted for in this country. The German firm did, however, manufacture and ship to this country in January, 1915, the cast-steel toothed racks for the piers and the toothed rims for the rollers.

The steel supports for the racks were furnished by the Minneapolis Steel and Machinery Co. These supports were drilled with $\frac{7}{8}$-in. holes and the $\frac{3}{4}$-in. holes in the rack were reamed to $\frac{7}{8}$-in.; the racks were then fastened to the supports with tight-fitting, turned bolts. The original design called for $\frac{3}{4}$-in. bolts. The two upper sections of the rack are not attached to the supports but are merely held in the pier in place by tight-fitting anchor bolts.

Erection of Roller Crests

The original program for the erection of the roller crests was as follows: (1) Construct a coffer-dam from pier *c* upstream to the main coffer-dam, and demolish the main coffer-dam between this junction and the east bank, or in front of spans *ab* and *bc;* (2) erect rollers *cd, de* and *ef,* and at the same time, by placing heavy rock with the cableway across the sluiceway channel, turn all the river through spans *ab* and *bc* and begin work in the sluiceway (i.e., finish concrete abutment *h* and pier *g,* place the paving in the tailrace and erect the sluiceway roller); (3) when the three rollers between piers *c* and *f* should be erected, raise them out of danger and demolish the coffer-dam in front of them, at the same time rebuilding the coffer-dam in front of spans *a* to *c;* and so soon as water could be turned from these spans erect the rollers upon them and finally demolish the remains of the coffer-dam. (See Fig. 1 for locations of sections noted.)



FIGS. 4 TO 9. CONSTRUCTION VIEWS OF THE ROLLER-CREST DAM, GRAND VALLEY PROJECT, U. S. RECLAMATION SERVICE

Fig. 4 (Upper Left)—Placing roller sections by cableway. Fig. 5 (Upper Right)—Placing the closing section of roller shown in Fig. 4. Fig. 6 (Middle Left)—Erection of roller BC, showing falsework suspended from truss beam. Fig. 7 (Middle Right)—Placing shields on the sluiceway roller. Fig. 8 (Lower Left)—Sluiceway roller in place. Fig. 9 (Lower Right)—End of sluiceway roller, showing track extension on the upstream side of pier, which takes the place of safety chains in the original design. Note also that the shield is boarded over

This program was essentially carried out except the erection of the sluiceway roller simultaneously with the rollers *cd*, *de* and *ef*. The increase in head resulting from damming the sluiceway caused such leakage through the coffer-dam as to endanger it; hence the diversion of the flow from the sluiceway was postponed until the three rollers *c* to *f* were erected and raised to a position where the flow under them would not interfere with continuing work upon them, and the coffer-dam protecting them would no longer be required.

The erection of the first roller was begun Mar. 25, 1915. This roller was shipped in five sections—the two ends, each about 13 ft. long, upon which the toothed rims were attached; the two adjoining pieces each about 21 ft. long and a closing section 6½ ft. long. The two end sections were swung into their approximate positions on timber falsework by means of the cableway, and with bars and jacks the proper teeth on the racks which had been set and the rims were brought into mesh.

The two adjacent sections of the roller were then joined to the ends and held in place by barrel pins and fitting-up bolts. The four sections were then adjusted for alignment at all parts of the circumference. The opening for the closing section was made about ¼-in. larger than the length required, and the slip or closing section swung into place. The end sections were then drawn together by pulling jacks and the splice-plates put on and held by barrel pins and fitting-up bolts.

The roller was next tested by wye level and the riveting commenced. When the riveting was completed on the roller proper, not including the shields and end shields, the roller was raised to give 8 ft. clearance over the ogee weir. It was here held in place by 12x12-in. Oregon fir posts, one under each end, strongly braced and fitting into a hole chipped in the bottom of the recess. This recess was then planked up to insure against displacement of the post by floating débris. In this position the shields were riveted to the rollers.

In erecting the roller *bc* a trussed beam of 8x18-in. Oregon fir was constructed and placed across the space between piers *b* and *c*, about 7 ft. downstream from the sill of the rollers. To this beam six cables were fastened and dropped down to support the downstream ends of the caps upon which the falsework was erected. Work on the roller *ab* was carried out in the same manner. In both instances extreme high water threatened the safety of the falsework and only quick work saved the situation.

The erection of the five rollers *a* to *f* was pushed with all possible speed, but during this period there were times when it was expedient to work on roller *fg*, which is east of pier *g* and inclosed on the opposite side by the sluiceway wall. It was blocked off by coffer-dams built both up and downstream from pier *f*. The apron below this roller was built around the sump to which drainage water was run during construction work. As soon as the sections of the roller had been assembled and riveted it was raised 8 ft. clear of the weir and the shields riveted in place. A temporary sill and sealing strips were attached and the roller lowered on the crest of the weir, effectually shutting off the water from this span and protecting the bay below and the coffer-dam above from pier *f* to the corner of the sluiceway.

The erection of the sluiceway roller did not begin until the early part of June, 1915. The sluiceway was cut off by three coffer-dams. The method adopted differed only slightly from that employed for erecting the 70-ft. rollers. The falsework for pieces Nos. 2 and 4 was built on flat cars running on a 36-in. gage track laid parallel to the sill, the flat apron of the spillway favoring this method. The erection and riveting of the sluiceway roller was carried out during a flood season, when the discharge of the river reached a maximum of 21,000 sec.-ft.

Paving Sluiceway and Slopes

The next operation was the concreting of the drainage sump and the paving of the downstream end of the sluiceway channel, after which the sluiceway roller was raised and the others lowered, diverting the entire flow through the sluiceway and finally demolishing the coffer-dams.

The heavy riprap below the downstream apron was placed with a 15-ton derrick, and a 36-in. gage truck was built along the spoil bank to bring the stone within reach of the derrick. This work was begun about midway between piers *f* and *g* and proceeded eastward. When the east abutment was reached the derrick continued downstream and also laid the slope paving downstream from the east abutment. Previous to placing the slope paving a footing wall 3 ft. wide and averaging 4½ ft. deep was built on solid rock. The embankment upon which the slope paving is laid is 10 ft. wide on top and forms the right-hand, or river side, embankment of the Orchard Mesa canal.

Operating Mechanism for Rollers

In March, 1915, steps were taken to have the chains and chain shafts made in this country. All except a sample set, which was made by the Vulcan Iron Works, Denver, Colo., are being made by the Lakeside Bridge and Steel Co., Milwaukee, Wis.; the contract price was $5,147 for the chain shafts and operating chains and $891 for the safety chains. Subsequently the part of the design calling for the safety chains was discarded in favor of triangular bents of structural steel, which form the upper part of the upstream roller tracks (see Fig. 9). The bids for the safety chains were rejected and the contract for the track extensions awarded to the Vulcan Iron Works, Denver, for $425. Since September, 1915, the rollers have been operated by shifting the single chain for the 70-ft. roller from one to another, and by temporary rigging from the cableway, in order to enable the diversion of water for seasoning and priming the canals.

The permanent power plant for the operating of the roller crests and canal headgates consists of a gasoline-engine-driven electric generator (25 kw.) which will supply current to six 10-hp. motors, operating the hoists for the 70-ft. rollers, one 20-hp. motor for the 60-ft. roller, and a 3-hp. motor for the headgates. A storage battery will also be installed, to be charged by the generator set. The battery will furnish enough power to operate one roller. The gears are also capable of being operated by a hand-winch.

The service bridge connecting the operating houses and intermediate piers consists of seven 3-hinge steel arch spans. Six are 70 ft. in length from pier wall to pier wall and one is 60 ft. The width of the bridge is 6 ft. inside the lower chords. The rise of the arches is, in each instance, 7 ft. and the curve is approximately para-

bolic. The floor is 3-in. planking at an elevation of 4,809.25, which is the finished elevation of the piers *a*, *c*, *e* and *g*. The latticed handrail on the upstream side is continuous from piers *a* to *h*. The service bridge was built by the Des Moines Bridge and Iron Works, Des Moines, Iowa, and cost 2.8c. per lb. It weighs 45 tons.

Conclusion and Acknowledgments

The elevation of the tops of the roller crests is 4,792.25 ft. For an assumed high-water level above the dam of 4,792.5 ft. the discharge over the dam and sluiceway, with all movable crests raised, is estimated at 50,000 sec.-ft. Besides this 2,000 sec.-ft. can be bypassed through the canals on each side of the river. The elevation of the tracks of the Denver & Rio Grande R.R. was raised from 4,792 to 4,797 ft.

It will be necessary at times to divert the whole flow of the river into the project canal. This requirement eliminated the possibility of using wickets or other inexpensive types of movable crest, on account of the leakage. The choice was therefore restricted to roller crests or large Stoney sluicegates. The greater ease with which the rollers could be operated was the determining factor. A roller was selected for the sluiceway gate on account of the lower cost and for uniformity in operating mechanism. The committee of Reclamation Service Engineers responsible for the adoption of the design is composed of R. F. Walter, D. C. Henry, J. H. Miner, F. Teichman and O. T. Reedy.

As previously noted Mr. Teichman was the designing engineer; Mr. Miner is project manager and O. T. Reedy was construction engineer in charge of erection until the summer of 1915, after which the project manager, Mr. Miner, was in direct charge. The chief engineer of the Reclamation Service is Arthur P. Davis, and Chief of Construction is F. E. Weymouth.

For the material of which this article is composed *Engineering News* is indebted to E. H. Baldwin, who was Acting Chief of Construction, and to Mr. Miner. The text is based largely on extracts from the project histories of 1914 and 1915 and on other data gathered on the site of the work by one of the editors.

Kansas Experience in Running Small Sewage Works

By F. M. Veatch*

SYNOPSIS—The need of and economy of efficient supervision of small sewage-disposal works is shown by Kansas experience with plain and two-story septic tanks, contact beds and their dosing devices.

The following observations are based on an investigation by the writer of the operation of sewage-disposal plants in the State of Kansas during a period of two years. It is believed that what may be said of Kansas plants is also applicable to those of other states.

The types of disposal plants found in Kansas are septic tanks, either ordinary or two-stage, contact beds, and one installation each of disinfection and of subsurface irrigation. Several sand filters and one trickling filter are under construction.

Ordinary and Two-Stage Septic Tanks

By variation of the length of the detention period of a tank, it may be operated either as a plain settling tank or as a septic tank. The greater number of our tanks are composed of two or more compartments which fact allows for some flexibility in operation, but a knowledge of the principles which underlie the working of the tanks is necessary for the successful adaptation of plant capacity to sewage flow.

All of our plants are subject to variation in sewage flow, due in some cases to leaky sewers or to fluctuations in the ground-water level, and in others to the uses to which the sewers are put. Therefore, it is easily seen that a plant cannot be expected to give the same purification through this range of conditions unless its capacity is correspondingly varied.

*Assistant Engineer, State Board of Health, Lawrence, Kan.

A point which has received little or no attention at our plants is the gradual change of capacity of tanks due to the accumulation of sludge. Owing to the wide range of conditions met with in different plants, it is impossible to set an optimum detention period for all plants, but this must be determined for each individual case, by experiment.

Many of our plants are so located that they receive their sewage in a very fresh condition, and in these cases there is no material benefit to be obtained by allowing the sewage to remain in a tank any longer than is necessary for the deposition of the greater part of the solids. The benefit to be gained by the sludge digestion that might take place by the use of the longer period is more than overbalanced by the increased difficulty of subsequent treatment either by dilution or on filters. In several cases the writer has found the effluent of tanks and occasionally those of contact beds, to require more water for their safe dilution than did the raw sewage, owing to lack of cleaning at the proper time or to improper detention periods. In such a case the plant is a detriment to a city and the money spent for its construction is a total loss so long as such conditions are allowed to prevail.

The advantage of the two-stage septic tank lies in the isolation of the digesting sludge from the settling sewage and to effect this the operation of the plant must receive daily attention. Gas vents should be kept open to prevent gas-lifted particles of sludge from entering the settling chamber. Scum should be removed from the surfaces of the settling chamber. The height of the sludge in the digestion chamber should be so regulated that it will at no time be nearer than a foot or so to the settling sewage. Sludge pipes and valves should receive sufficient attention to insure their readiness for operation at all times.

The extreme offensiveness of the collections on screens and the danger of backing up of the sewage make their cleaning a necessity as often as they become clogged, since a poorly operated screening chamber is worse than useless.

Dosing Chambers and Devices

Failure to flush dosing chambers has frequently been a source of trouble in our plants, and the subsequent treatment of good tank effluents has on this account been made more difficult. Holding the effluent in contact with septic sludge in a dosing chamber has the same or worse effect than an equivalent lengthening of the detention period in the tank. We have increased the efficiency of some of our contact beds by changing the piping of the dosing siphon, so as to allow the tank to discharge at a lower level, thus decreasing the period in the chamber.

No matter what the design of a dosing device may be, it must be thoroughly understood by the operator, in order that small repairs and adjustments may be quickly made, when necessary.

When seen for the first time any device seems to be merely a maze of piping and it is not surprising that much trouble has been encountered in their operation by men who do not understand their construction. The writer has been called upon to put a dosing apparatus into operation when all that was necessary was the correct adjustment of a three-way cock. Every plant operator should have a setting plan of his device and should be so familiar with it that he is able to vary its operation at will. Such drawings may be had free of charge from the manufacturers of the apparatus.

Where secondary treatment is practiced, the importance of the successful operation of the dosing apparatus cannot be overestimated. Time after time the writer has visited plants, after having been assured that they were operating successfully, only to find the dosing device out of order and the whole plant out of commission. Any device will at times fail to operate, but if it is immediately attended to no permanent injury results. Several cases have been noted by the writer where a contact bed has been practically ruined by the failure of an alternating siphon to take its seal, thus allowing the sewage to run through one bed continuously.

Need of Care in Operating Contact Beds

Treatment on contact beds cannot be successfully practiced without much care in the adjustment of the operation cycle to the rate of flow of sewage. This cycle must be selected with due regard for the character of the tank effluent treated, as well as for the rate of flow, and can only be chosen by trial in each plant. In general, the results of our beds have been the best in cases where relatively fresh tank effluent has been treated and the writer has yet to see a stable effluent produced by a plant which treats the effluent of a tank with a detention period of over twelve hours.

Cases have been found where no attempt has been made to set the valves in the timing chambers so that each bed will have an equal contact period, and in such cases no efficiency need be expected.

Lack of protection from back water has proved a source of trouble in several of our contact beds. This has happened in plants well protected by levees and in many cases has been due merely to the failure to close an outlet valve and bypass the sewage at times of flood. A single flood from one of our turbid Kansas streams will do more toward clogging the stone in a contact bed than several years of careful operation.

The length of time that a contact bed will operate successfully, without cleaning, is determined largely by the care it receives. We have beds that have been in operation for eight years and are still in good order, but we also have others that have become so clogged in the course of a year or so that they will have to be renewed before a good effluent is produced. The cleaning of contact-bed material is a very expensive proposition and the economy of efficient supervision is very plainly shown by the decreased cost of upkeep due to the increase of the period between cleanings.

Amount of Supervision Required

It is not intended to recommend the employment of an expert for the handling of each plant. In Kansas and many other states this is attended to by the State Board of Health, which maintains an engineering department, the services of which are at the disposal of plant operators who wish help or instruction. Furthermore, the entire time of one man will hardly be required for the work but an average of three hours each day will in most cases more than suffice for the routine work. In practically every case it is possible to give this man enough other city work to fill in his time. More help will of course be required when units are cleaned.

The duties of the plant operator should consist of: 1. The making of a daily trip to the plant. 2. Making sure that all parts of the plant are in working order and in operation. 3. Making a few simple tests to determine the amount of sewage being treated, and the character of the effluent. 4. The recording of his results in a systematic log.

It has been stated that the services of an expert need not be secured. This is true but it is not meant that the work of any common laborer will suffice. The man should be intelligent, quick to learn and above all he must be a willing worker and take an interest in the plant.

The necessary tests for the operation of the ordinary small plant are: 1. Flow gaging. 2. Relative stability. 3. Settling solids.

Importance of Flow Gagings and of Tests

The rate of flow through any plant must be determined before it can be intelligently operated. A permanent weir is probably the most satisfactory equipment for this purpose, although in some plants good results may be obtained by noting the rise in the dosing chamber. Gagings should be made throughout an entire day at intervals of one hour or less every month or so in order to determine the exact daily flow, and the hourly variation. If this is done the operator may calculate the approximate daily flow from his hourly variation curve. A better method would be to use an automatic recording gage in connection with the weir. The first cost of such a gage is high, but the time saved soon pays for it.

The relative stability test as outlined in the standard methods of the American Public Health Association is very easily made and is recommended to the small plant operator. It will be found at times that effluents must be diluted with water before the test may be made and in this case it will not, strictly speaking, be the relative stability that is tested for, but rather the amount of dilution required to produce a certain relative stability.

The use of Imhoff cones or settling glasses as an aid to the operation of tanks and contact beds is recommended to the operators of our plants. While these glasses may not give results comparable with the actual suspended solids test, much practical good may be derived from their use, as a check on plant operation.

Economy of Proper Supervision

In the above statements, the production of a good effluent has been emphasized as a reason for efficient supervision and the subject treated from the standpoint of the sanitary engineer. However, when the subject is analyzed with dollars and cents as a basis the economy of supervision is none the less evident. In the first place, a sewage-disposal plant represents an investment of several thousand dollars, the interest on which is represented by useful work by the plant in the prevention of nuisances. In the second place, a plant is composed of parts subject to more or less rapid deterioration, the useful life of which depends on proper care. Therefore the city that allows its plant to work below maximum efficiency is suffering loss just as surely as the money lender who is unable to collect interest on his loans.

Examples of loss by the above means have often been noted in our plants. In one instance an expensive apparatus was installed for the treatment of septic tank effluent with liquid chlorine. This machine gave good results during a two-months' test by the writer, but less than four months later he found, on an inspection trip, that the machine had been allowed to operate with a leaky union on the chlorine feed pipe, and as a result was in such a state of corrosion that its practical renewal was necessary. In this case a lead gasket and a very few minutes of time would have saved the city an amount equal to the cost of supervision for several months.

In another case, that of a plain septic tank, a sludge-pumping outfit was installed when the plant was built. This outfit consisted of a 6-in. centrifugal pump driven by a 20-hp. gasoline engine. The tank was constructed so large that up to the time of the inspection (four years) no sludge pumping had been done. At present the engine is in such a state, owing to misuse by trespassers and lack of attention at times of high water, that it is doubtful whether it can ever be used and it is certain that an expenditure of a considerable sum of money for labor and materials is necessary before sludge pumping may be done. This case represents an investment of over $1,000, from which no return has ever been realized. Other like instances might be cited but the above are representative of this phase of the problem.

Too often a city considers that its obligation is discharged by the construction of a sewage-disposal plant, and fails to realize that its upkeep is just as important a consideration as is its construction. There is also by far too great a tendency toward resting the blame of poor results from sewage-disposal plants upon the shoulders of the designers.

In Kansas, at least, much of the present unfavorable criticism of the practice of sewage disposal would be quieted if each individual plant was so operated that its maximum efficiency could at all times be obtained. As every one familiar with the subject knows, the practice of sewage disposal has undergone great changes in the past fifteen years, and, as a result, some of the earlier plants are somewhat obsolete. This, however, should not be made the criterion by which plants are judged.

Concrete Hydrant and Valve Chambers, Salt Lake City

The city engineering department of Salt Lake City, Utah, which is headed by Sylvester Q. Cannon, has a standard setting for fire hydrants and their valves which makes all parts of the hydrant accessible.

Instead of planting the fire hydrant in the ground with a separate valve box in the middle of the street, as is

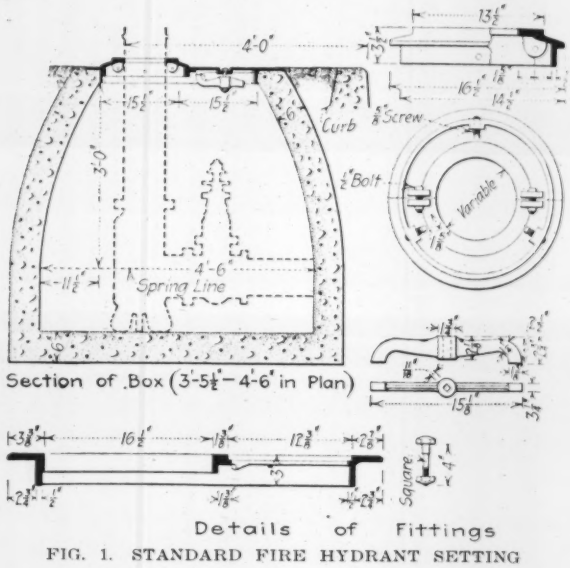


FIG. 1. STANDARD FIRE HYDRANT SETTING

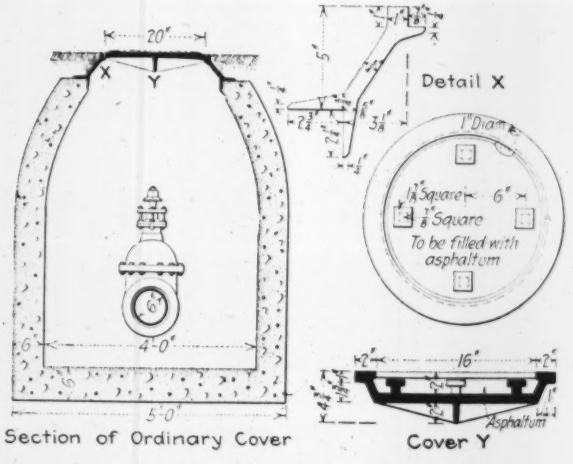


FIG. 2. STANDARD VALVE SETTING, SALT LAKE CITY

common practice, both the hydrant and valve are enclosed in a concrete chamber as illustrated herewith.

No trouble has been experienced in Salt Lake City from freezing on account of the exposed connection of the fire hydrant, but temperatures there are not extreme.

The hydrant stands 4 ft. back from the curb, so that the valve box is entirely under the sidewalk. This is possible in Salt Lake City because of the very wide streets and sidewalks. In many cities a hydrant so placed would be a serious obstruction to traffic.

The standard water-works valve chamber, for valves from 6 to 12 in. in size, is shown by Fig. 2. A similar box but of larger dimensions is used for all water-main valves. Two covers are shown, a plain iron casting and an asphalt-filled casting for use on asphalt-paved streets.

Water-Power Storage Reservoir with Pumping Station

SYNOPSIS—In connection with the use of Bear Lake, Utah, as a storage reservoir for the control of Bear River and the conservation of water from the upper reaches of the river, a unique project has been developed. Bear Lake lies in the lower reaches of Bear River, there being 3,000 sq.mi. of drainage area above. The storage capacity of the lake is large relative to the annual stream flow. To restore the impounded water to the river the pumping station is being built with five 300-sec.-ft. units, the largest for as great head. Construction problems brought about by local topography and some details of equipment are given.

A pumping station of 1,500-sec.-ft. capacity is being built by the Utah Power and Light Co., at Bear Lake, 100 mi. northeast of Salt Lake City, in connection with the development of this lake as a storage reservoir for the control of flow in Bear River and the conservation of all of the water in the upper reaches of the Bear River drainage area for use in both power and irrigation.

As shown in Fig. 1, Bear River flows down past Bear Lake about 5 mi. distant. The storage capacity of Bear Lake is large relative to the total annual stream flow. There was originally a small natural outlet of insignificant capacity from the lake basin to the river, but it was some 20 mi. long and crossed a very flat swamp. During the high river stages of spring, water sometimes flowed back from the river. The pumping station is to lift the stored water for discharge back into Bear River without the necessity of expensive works for a gravity channel draining the lake. The station will be operated by electric power developed on the stream regulated.

The Utah Power and Light Co. obtains its power from hydro-electric stations, the largest and most important of which are on Bear River. There are four stations of 67,500-kw. total capacity utilizing the same water through a total head of 865 ft.

Bear Lake Storage Project

Bear Lake is a deep body of water approximately 21 mi. long and 8 mi. wide. At the north end there is the large swamp, already referred to, and a shallow lake called Mud Lake, which is 6 mi. long and 5 mi. wide. The inlet canals through which water is diverted from Bear River discharge into the north end of Mud Lake, and the water then runs 6 mi. before coming into Bear Lake. When Bear Lake has been pumped down over 5 ft. practically all the bottom of Mud Lake will be exposed. In Bear Lake proper the shores are very steep and there is relatively little difference in area between the high and low water levels. One foot of drawdown of the lake equals 68,000 acre-ft. The total capacity available is approximately 1,300,000 acre-ft., or 57 billion cubic feet. The drainage area of Bear River above Bear Lake is approximately 3,000 sq.mi.; the drainage area of Bear Lake proper is approximately 400 sq.mi. The capacity of the inlet canals is 5,000 sec.-ft., so that practically all flood waters can be diverted into Bear Lake.

Irrigation is quite extensively carried on in the Bear River drainage area, the water being diverted directly from the main river and also from a great many of its small tributary streams. The regulation of the river flow for power purposes automatically regulates the flow for irrigation also, and besides preventing a shortage of water in dry years on the land now under cultivation, will provide water for irrigating large additional areas, both by gravity and electric pumping distribution.

A canal has been dredged through the north swamp from a point called Lifton on the north end of the lake

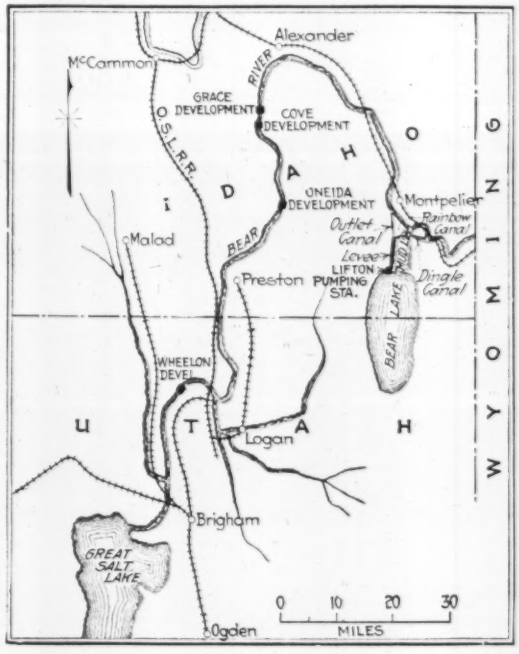


FIG. 1. MAP OF BEAR RIVER POWER SYSTEM

14 mi. north to Bear River at Junction. When the lake is full there is approximately 8 ft. fall between the surface of the lake and the surface of the river at Junction. Almost all of this fall is necessary for the grade of the canal for the maximum required capacity. It is possible to draw the water from the lake by gravity to a depth of approximately 4 ft. by means of this outlet canal, but the quantity of course reduces as the lake level goes down and the hydraulic grade of the canal flattens out.

The pumping plant will make available for storage purposes all the water in the lake for a depth of practically 21 ft. During a year of average flow, the variation in the lake level necessary to equalize Bear River just below the reservoir would be approximately 3.5 ft. The large capacity of the reservoir will make it possible not only to equalize the stream flow throughout an average year, but it will make it possible to equalize the stream flow throughout any series of extraordinary dry years of which there is any record.

Past records indicate that so to equalize the flow of the river only 12 ft. of the depth of the lake is necessary However, the pumping station has been designed so that

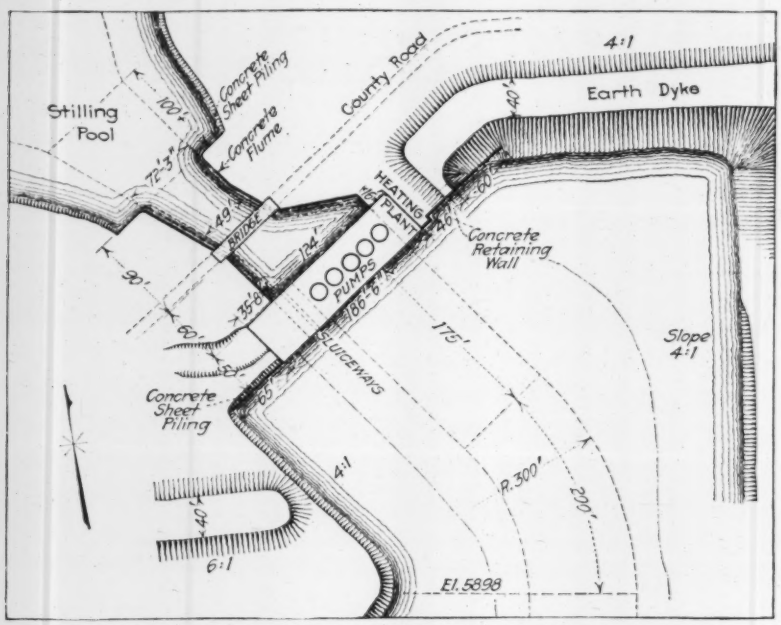


FIG. 2. LAYOUT OF LIFTON STATION

the use of 20 ft. of storage will be possible, in order to provide for errors and inaccuracies in the data, and also to make it possible to use the storage reservoir to equalize the power output from streams other than Bear River. Such equalization of power output will make more water available for irrigation during the summer months than would be available if the stream flow were equalized throughout the year. Just how far down it may be desirable at some time in the future to pump the water in the lake is problematical, and, since the additional cost for placing the suction of the pumps a few feet deeper is very small, it was decided to build the plant with several feet of additional capacity available.

It was necessary to locate the pumping station on a sand bar called "The Causeway," which extends entirely across the north end of the lake and separates Bear Lake from Mud Lake and the swamp. The material all about this locality is a very fine sand mixed with clay, and this extends to an unknown depth. The only variation is in the percentage of sand and clay. This material flows so readily in water that the wave action against The Causeway has flattened the material out so that the bottom of the lake where it is 20 ft. deep is approximately 2,000 ft. from the north shore. It is planned to dredge a large channel through this beach to the pumping plant, which is located approximately 150 ft. away from the shore line of Bear Lake. The station is situated back from the shore line on account of the fact that heavy windstorms sometimes break up the ice on the intake and drift it up on The Causeway to a height of 10 or 12 ft. For this reason the pumping plant has been put on the side of the channel, and if large quantities of ice are blown into the channel the ice will slide up on the shore at the end, where there are no structures.

It is expected that the maintenance of this channel from the lake to the pumping station may be quite a difficult problem, as the action at the mouth may gradually fill it up with sand. The construction of a permanent intake channel would involve such large expenditure that it was decided to maintain the channel by dredging it very large at first and by additional dredging in the future as might be found necessary. On account of the peculiar foundation conditions, the building is built on a concrete mat supported on piles. In order to place this mat and unwater it, an open wooden caisson 140x60 ft. and 44 ft. deep was used. This caisson, which is by far the largest yet built in the world, will be described in a future article.

Pumping Station Equipment

The pumping station will contain five vertical-shaft units, each having a nominal capacity of 300 sec.-ft. The head will vary from nothing to 20 ft., and variable-speed motors and two types of impellers have been provided for best operation at both high and low heads. The accompanying sketches show the arrangement of the principal and auxiliary apparatus in the station.

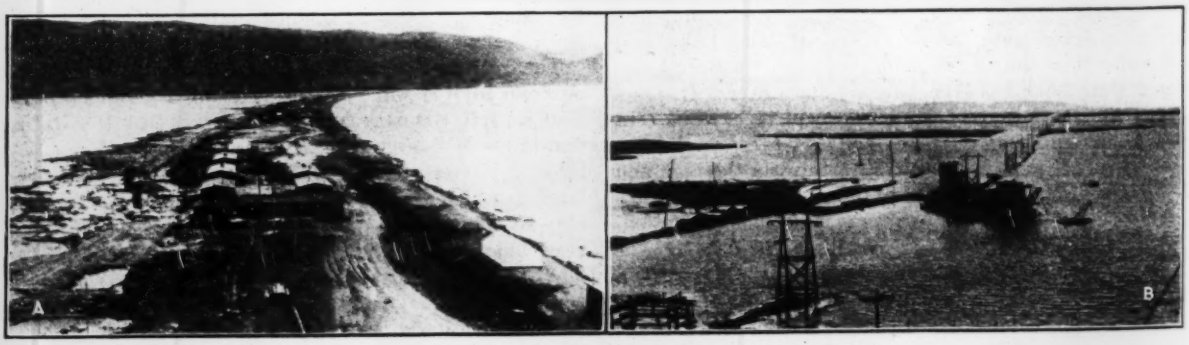


FIG. 3. TWO VIEWS OF WORK AT BEAR LAKE, UTAH
A—Looking east along the Causeway. B—Looking north along the discharge canal. Both views from top of concreting tower

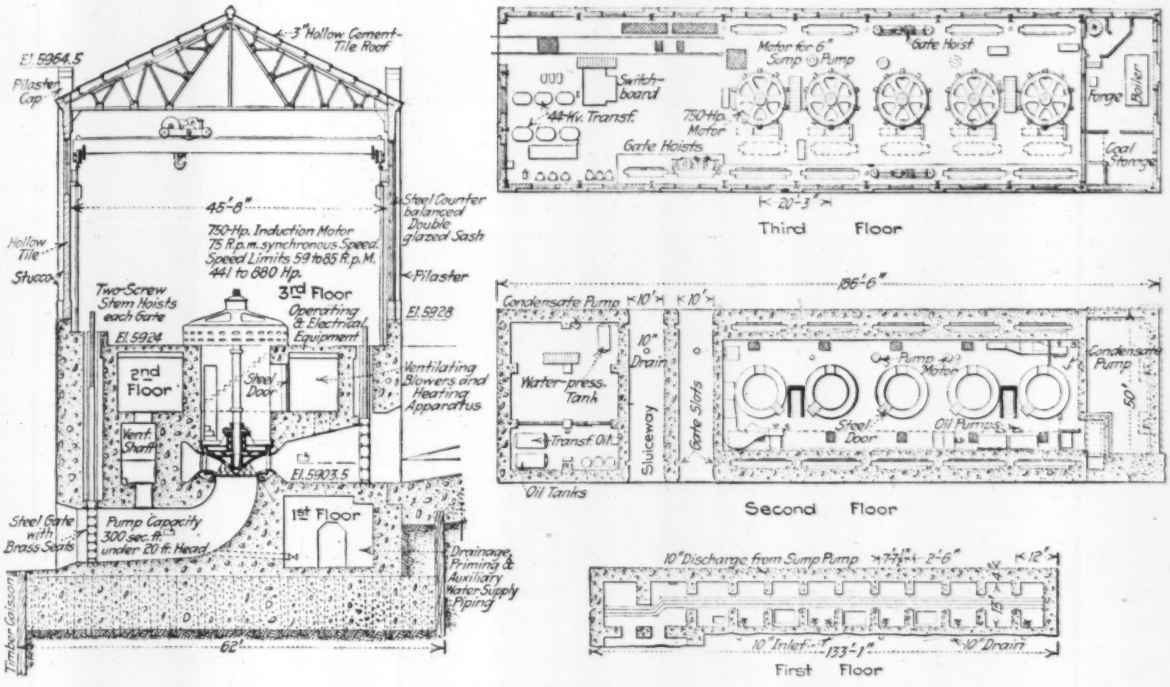


FIG. 4. SECTIONAL PLANS AND ELEVATIONS OF LIFTON STATION

The pumps in the Lifton Station have, so far as known, the largest rate of discharge for as high head—300 sec.-ft. (134,500 gal. per min. or 194,000,000 gal. per day) against 16 ft. and running at 77.5 r.p.m. The construction, as shown in Fig. 5, is very similar to recent vertical-shaft single-runner Francis-turbine units. The casing and suction tube are molded in the concrete foundations. The vanes around the impeller are not diffusion vanes as they conform to the free path of the flow; they are cast solid with the foundation ring supporting in part the concrete of the casing and the motor above. The steady bearing is of the water-lubricated lignum-vitae type, and is held by a diaphragm immediately above the impeller. A Kingsbury thrust bearing, mounted on the motor bracket, carries the revolving element of each pump.

All five pumps are alike and are designed to draw the lake down 20 ft. Normal full load speed of motors is 72.5 r.p.m., at which speed the pumps will have a capacity of 300 sec.-ft. up to 13 ft. head. It is planned to install speed regulators later which will give a speed range of 58.4 to 82.9 r.p.m. These regulators will give each pump a capacity of 250 sec.-ft. for the maximum operating head of 20 ft. At low heads, the pumps will be operated at a speed lower than normal, for economy in power consumption and also for controlling the capacity. A second type of runner has been designed for use in these pumps for reducing the power consumption at very low heads. Up to and including 5 ft. head, this second type will have a capacity of 300 sec.-ft., and will have reduced capacities up to 10 ft. The designs for this development were prepared in the engineering offices of the Electric Bond and Share Co., of New York City. The direct supervision of construction has been by the Phoenix Construction Co., of Salt Lake City. The pumps were made by I. P. Morris & Co., of Philadelphia, and the motors and electrical equipment by the General Electric Co., of Schenectady. As stated before, a description of the design and construction of the large caisson foundation for the pumps will be published in an early issue.

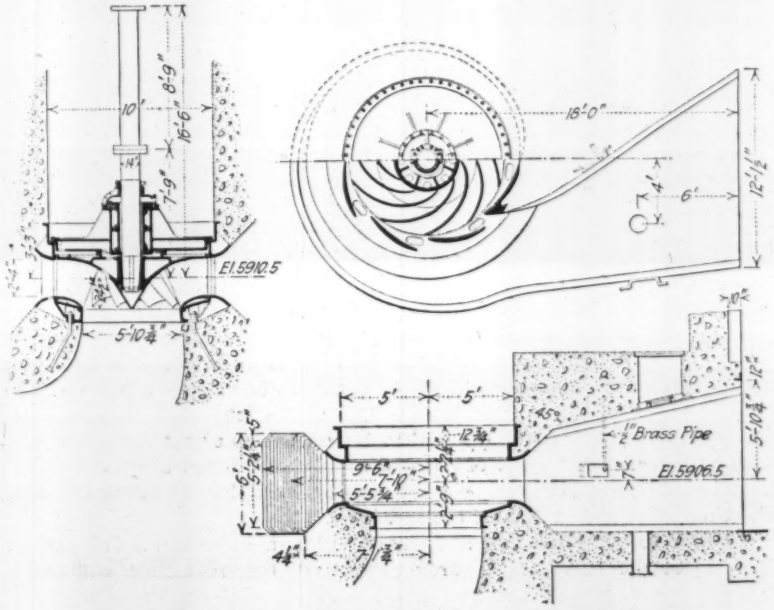


FIG. 5. DETAILS OF LIFTON 300-SEC.-FT. PUMP

Details of Roof-Arch Collapse in Atlanta

The roof of a building on S. Pryor St., Atlanta, Ga., used as a skating rink, collapsed and fell about 9:30 p.m., June 9, killing three people and injuring more or less seriously about ten more. Two of those killed and most of the injured were caught under the falling roof; one of the employees of the rink was struck by falling brick from the broken front parapet wall as he climbed through a window. The building has been used to stage boxing contests, and one drawing a large crowd was held there recently. The cause of the accident has not been determined by the city authorities.

The building was 101 ft. wide and 165 ft. long, facing east on Pryor St. It was built about a year ago, and the lumber used in its construction seems to be in good condition. No plans of the building are available, as the city does not require the filing with the building department of plans of one-story structures. The architect who designed it (G. W. Foote) has retired since that time, and all his less important plans, of which he considered this one, were destroyed for lack of room.

The accompanying sketch shows the details of design as well as could be gathered from the trusses themselves. All the lumber seems to be merchantable-grade Southern yellow pine. The dimensions given on the sketch are for rough lumber, while all that used is finished on four sides, making the dimensions less by $\frac{1}{8}$ to $\frac{3}{8}$, depending on the thickness, etc. The walls of the building are something over 20 ft. high of 3x4x8¼-in. vitrified paving blocks laid in lime mortar. The front wall and the first 100 ft. of each side wall are 16 in. thick; the remainder of the side walls and the rear walls are 8 in. thick, with 8x20-in. pilasters spaced 11 ft. center to center.

The roof trusses, spaced 11 ft., are of an arch design, rising sharply from the floor at the ends (batter about 3¾ to 12), and are nearly flat for the larger middle portion of the span. The boards of which the top and bottom chords are composed are laid flat (Fig. 4) and nailed, about one 8d. nail per foot showing on top. The radial and diagonal struts are in two planes, as shown (Figs. 2 and 3), the diagonals alternating from panel to panel. The ends of the trusses rest on concrete footings about 1 ft. below the level of the main skating floor. There is an observation floor running entirely around the building, elevated about 10 in. above the main floor, and at the rear there is a second floor or platform 10 in. above the first. There were no fixed benches or seats, but a number of chairs were placed promiscuously along the observation floor. At the rear of the hall there is a pile of temporary benches used during the recent boxing match and now stacked up out of the way.

The roof was nearly flat, with a light pitch toward the front of the building so as to carry the rain water to city drains in the street. It was a light-weight roof of felt, tar and gravel laid on 1-in. sheeting nailed to 2x6-in. purlins, which in turn were supported on 4x6-in. rafters running directly over the trusses. The rafters were supported on 4x6-in. posts about 11 ft. apart, which stood on top of the roof trusses. The pitch of the roof was given by varying the lengths of these posts, as the trusses were all set at the same elevation. The trusses were held together by 2x6-in. ties about 10 ft. center to center, nailed flat on top of the top chord with five or six 20d. nails at each connection.

The ties were ordered put in by the building inspector after the construction started. The posts were toe-nailed to the tops of the trusses and the rafters toe-nailed to the posts, using four or five 10d. nails at each connection. The purlins were toe-nailed to the rafters, using one 8d. nail at each rafter. Where the ends of the purlins met, if they lapped sufficiently, they were nailed together with one 8d. nail and each of them toe-nailed to the rafter.

The purlins were stock 22-ft. material. What lap they had, if any, was whatever extra length the lumber chanced to have, that is, there does not appear to have been any intention to tie the purlins together at the ends with the object of making them continuous. In some places the lap was as much as 4 in., generally less and sometimes almost none at all. All purlins were laid with even joints and the sheeting was laid in similar manner, making continuous joints along certain purlins (foreground of Fig. 2, background of Fig. 4). Running along the middle of

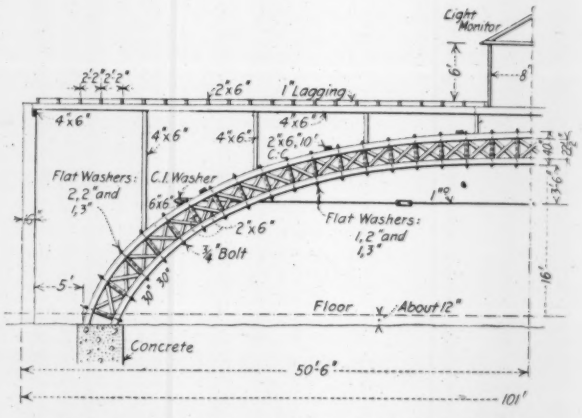
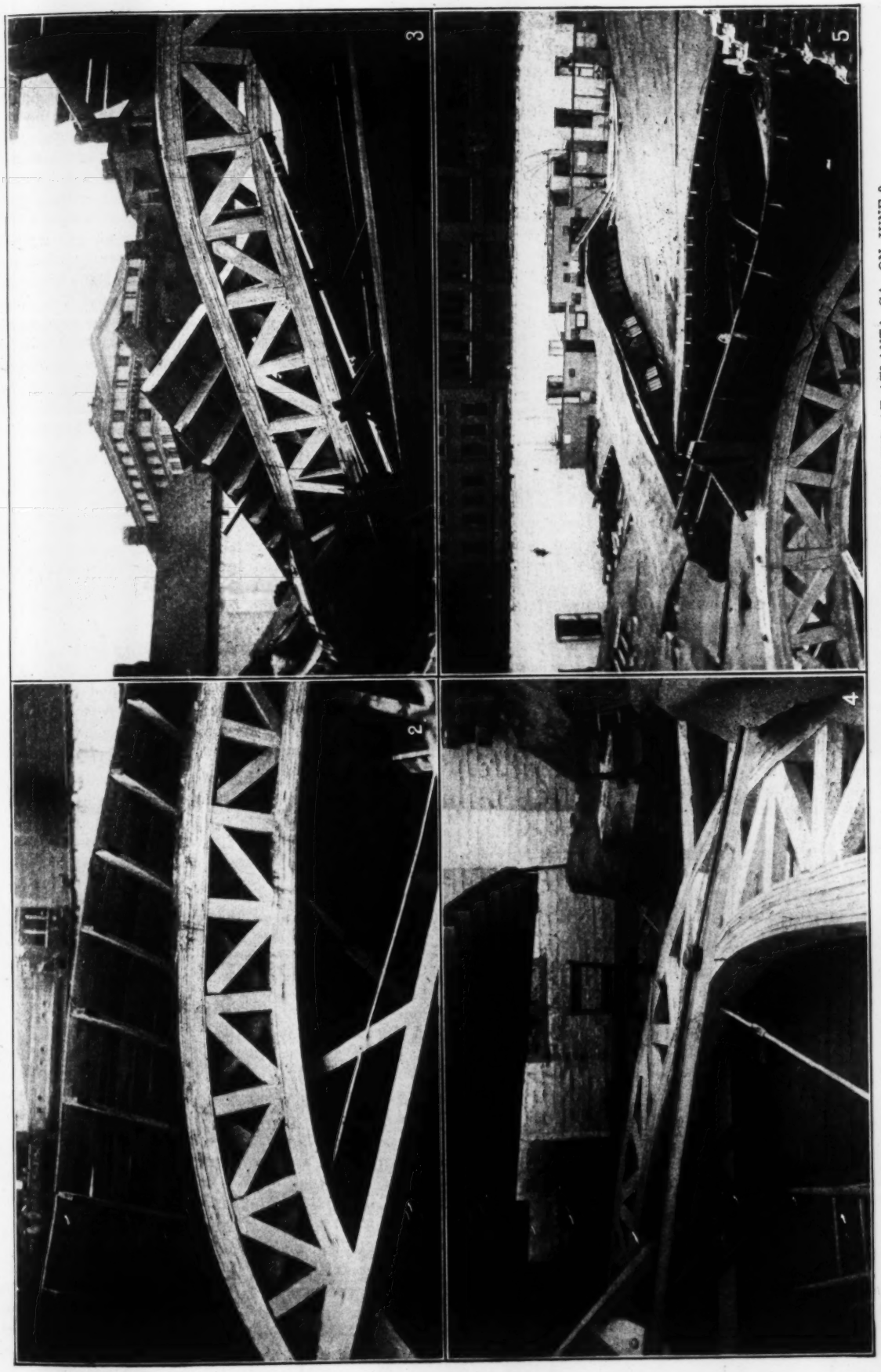


FIG. 1. DETAILS OF ONE OF THE ARCHES

the building on top of the rafters from about 30 ft. from the front to about 45 ft. from the rear was a monitor of light construction, mostly windows, added to the original design after the construction was under way (Fig. 5).

The roof trusses were designed, built and erected without the tie-rods. During the erection there was a hard rain, and water several inches deep was impounded on top of the roof immediately to the rear of the skylight, showing that the trusses must have sagged under their own weight and the weight of the roof above them. They were immediately jacked up and propped and the ties put in to hold them up. (The building inspector says he ordered them in; the architect says he put them in of his own accord.) Only two of the trusses sagged under the weight of the water, and they were strengthened with additional braces in the form of a ¼x12-in. plate several feet long fastened by lagscrews to the side of the arch at the haunch at each end and a long plate of the same strength, fastened to the other side of the truss, reached from one of these shorter plates to the other and overlapped a little. A similar long plate is screwed to the rear side of the last truss.

The forward truss was 22 ft. in from the front wall. At 11 ft. from the wall the roof was supported on 2x4 rafters nailed to 4x4-in. posts, which were set on the top of a light wooden partition wall about 8 ft. high,



FIGS. 2 TO 5. VIEWS OF THE TIMBER ARCH TRUSSES OF SKATING RINK THAT COLLAPSED AT ATLANTA, GA., ON JUNE 9.

separating the offices, dressing rooms, etc., from the main hall.

The workmanship generally seems to have been fair, though the jointing of the diagonals of the trusses was not exact.

All the arched trusses fell toward the rear of the building and lie, lapping over one another, with the feet near where they stood. Only two of those visible are broken—the rear truss, in two places, and one near the middle of the building, in one place. Both were broken sidewise from striking obstructions as they fell. The rear truss is partly supported by a small room at each end, which reaches from the truss hard up against the rear wall, the room walls being about 8 ft. high. It might have been the push of the trusses transmitted through the 2x6 ties and the wall frames of these rooms, aided by the blow of the falling truss nearer the top of the wall, that caused the rear wall to fall.

It fell outward against a bank, almost as a complete wall, breaking loose near each end (one end shown in Fig. 5) and practically at the floor level. Parts of the wall, including the pilasters, are still intact. The window frames lean up against the bank, with the brick still in place above the frames.

The roof fell practically vertically. The condition of the glass in the monitor windows would seem to indicate that it fell all at once, producing litle strain in the monitor frame. The roof parted at most of the purlin joints and at some of the sheeting joints (Fig. 5). The first 22-ft. section at the front of the building stayed practically in place, supported by the posts on the partition wall. It was removed next day by order of the building department. Apparently this section was pulled down a little way at the rear end, as a part of the concrete-coped brick parapet along the front of the building was pried off and fell on the sidewalk. Six or seven courses of the tops of several pilasters along the south wall were pulled off by the plate as the roof fell. The pilasters generally are tied to the walls every header course, but the tops of these particular ones were not tied.

There are a few things about the design of this roof that deserve notice: The roof trusses apparently were not able to support the weight of the roof as originally constructed, so that reinforcing ties were put in after the trusses were up and the roof on. There was no sway bracing—nothing to prevent the trusses from falling over, once they overbalanced their less than 10-in. foot. The 2x6-in. ties later put in by order of the building inspector helped steady individual trusses, so that they must all have acted together, at least to the extent of the strength of the nails holding the ties. The posts supporting the roof above the trusses stood unbraced on top of the trusses, so that if the truss got out of plumb or the post itself were not exactly vertical, the weight of the roof above would have a tendency to push them over. The purlins were all laid with even joints, so that the roof did not have the stiffening that broken joints would give and parted in sections when the roof fell. The sheeting was similarly laid with a similar result.

Curves for Irrigation-Ditch Velocity and Discharge

By L. R. Douglass*

Recently the writer was called upon to cross-section and measure a ditch some five or more miles in length, with a view of accurately determining the maximum velocities and discharges. Cross-sections were taken every 100 ft., and oftener where thought necessary. In order to eliminate the long and tedious calculation of Kutter's formula and also to save time and allow for a more intelligent study of each varying section, the accompanying diagrams were evolved.

Their use can be most easily explained by example. Take a ditch whose top width is 12 ft., bottom width 10 ft. and depth 2 ft. The grade or slope is 5 ft. to the mile, or for convenience 5 ft. in 5,000 ft. By calculation the wetted perimeter is found to be 14.47+.

*Of Danford & Douglass, Civil and Mining Engineers, Trinidad, Colo.

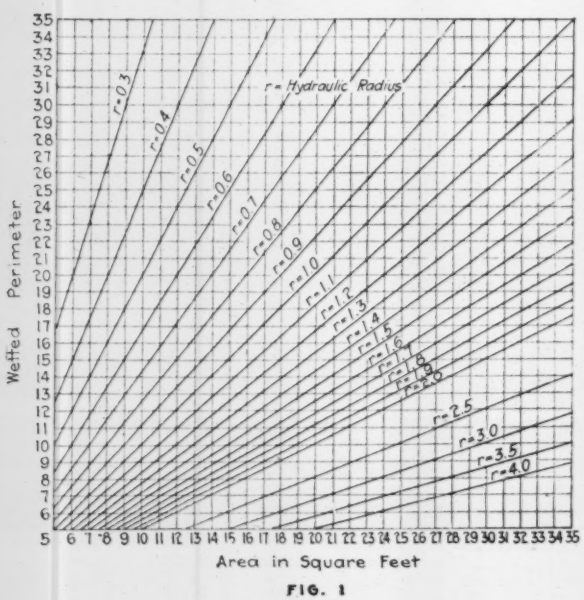


FIG. 1

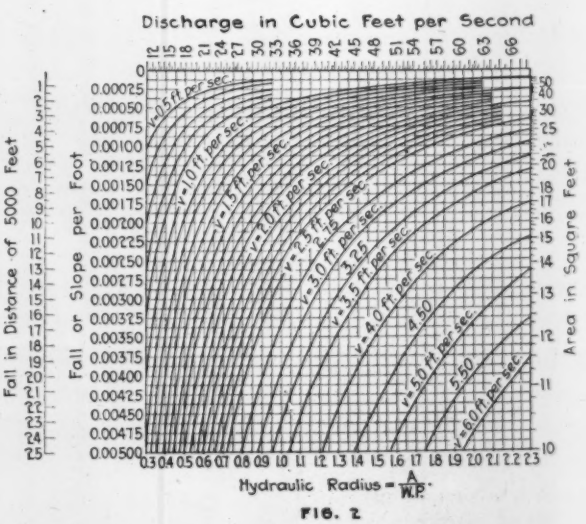

Discharge in Cubic Feet per Second
Fall in Distance of 5000 Feet
Fall or Slope per Foot
Area in Square Feet
Hydraulic Radius = A/W.P.
v=0.5 ft per sec.
v=1.0 ft per sec.
v=1.5 ft per sec.
v=2.0 ft per sec.
v=2.5 ft per sec.
v=3.0 ft per sec.
v=3.5 ft per sec.
v=4.0 ft per sec.
v=5.0 ft per sec.
v=6.0 ft per sec.

FIG. 2

CURVES FOR FINDING VELOCITY AND DISCHARGE OF OPEN CHANNELS

In Fig. 1 find the area 22 at the bottom and run vertically upward until an intersection is made with a line produced horizontally from the wetted perimeter of 14.47. The hydraulic radius of 1.52+ is thus determined.

The next step is to find the velocity. Proceeding with Fig. 2, we find the hydraulic radius of 1.52 at the bottom of the sheet and run vertically upward to an intersection with the slope line of 5 ft. in 5,000, or slope of 0.001 per ft. This intersection point lies between the velocity curves 2.1 and 2.2 ft. per sec. By proportionally scaling (if greater accuracy than interpolating by eye is required) we find the intersection point to be at the velocity of 2.18 or 2.19 ft. per sec. Having the velocity, the discharge is then determined by taking the area of 22 at the right of the sheet and producing it horizontally to the velocity line of 2.18. From the point thus found, run vertically upward to the discharge table at the top of the sheet, and the resulting discharge is found to be 48 cu.ft. per sec., as nearly as can be read.

The velocity curves are based upon the value of $n = 0.027$ in Kutter's formula. This value is used where the canal is more or less irregular in regard to alignment and cross-section, but free from stones and weeds. Such conditions almost always exist in the small irrigating canal ranging in discharge from 15 to 60 cu.ft. per sec.

With regard to the range in the velocity curves, it may be well to state that in this country, where the soil is mostly a sandy loam, velocities of 1.5 and 2.5 ft. per sec. may be considered as limiting: The first, in that it is sufficient to prevent deposit of silt; and the second, because it is slightly below that which would cause excessive erosion. The curves without this zone, shown in Fig. 2, were constructed to cover conditions as found in the ditch previously mentioned and have no particular value except for like cases.

Actual calculation by Kutter's formula with n as 0.027 gives a velocity of 2.202 and a discharge of 48.44 cu.ft. per sec., for the example assumed in order to illustrate the use of the table. By increasing the size of the table and the number of velocity lines or curves, it is readily seen that results can be obtained to a correspondingly greater accuracy.

Platform Shelters of Concrete Unit Construction

The new station of the Southern Pacific Co. at Los Angeles, Calif., has the platforms covered by shelter roofs built of reinforced-concrete units. The roofs are of the "butterfly" or concave type, sloping down from the eaves to a central gutter. There are four platforms 740 ft. long and 16 ft. wide, with roofs 18 ft. wide.

The main construction consists of a central row of T-head columns, spaced 20 ft. c. to c. and carrying two rows of roof slabs 20 ft. long. This is shown clearly in Fig. 1, with a locomotive crane placing the slabs in position. The columns are 12x24 in. at the base and 12x18 in. at the top, with caps or heads 10 ft. long and 17 in. deep. On each side of the cap is a recess, forming a shelf for the end of the roof slab.

Reinforcing bars projecting from the ends of the slabs lap each other over the top of the cap and are embedded in cement mortar, which is filled in over the cap to the level of the slabs. The longitudinal joint between the

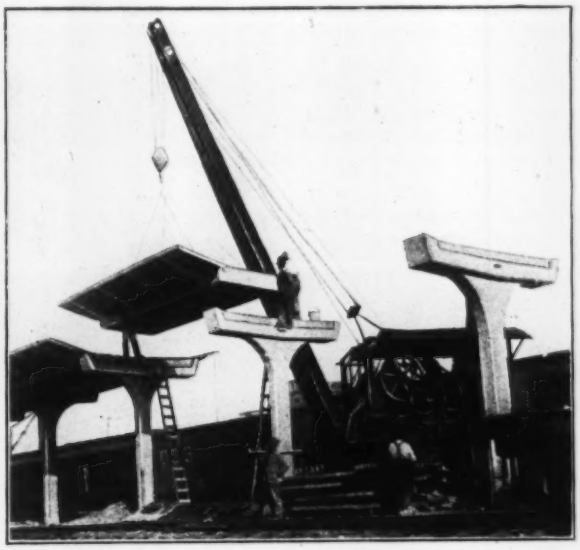


FIG. 1. PLACING ROOF SLABS OF PLATFORM SHELTERS BUILT OF CONCRETE UNITS; LOS ANGELES, CALIF.

slabs is also filled with cement mortar. A roofing composition is laid upon the completed shelter. At intervals of about 340 ft. an expansion joint is provided, covered with copper flashing.

The roof slab has a minimum thickness of 4 in. (increased to give an incline for drainage) and is stiffened

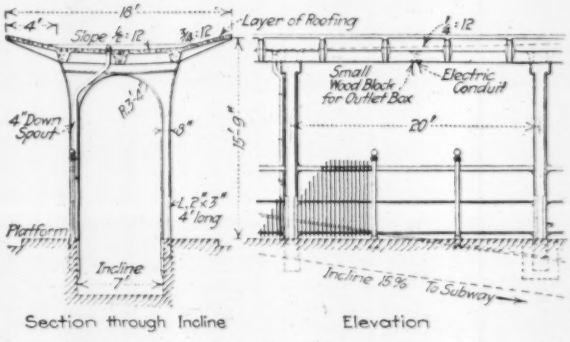


FIG. 2. CONCRETE PLATFORM SHELTER WITH TWO-POST BENTS SPANNING INCLINE APPROACH

by six transverse ribs. The columns are set in sockets, 18 in. deep, in pedestal footings.

A variation in this construction is required where the inclined ramps connect the platforms with the subway beneath the tracks. Here the roof is carried by two-column bents, as shown in Fig. 2. These columns are 8x12 in., connected by an arched cap. They are set in steel sockets in the steel framing of the subway roof.

This station was designed and built under the direction of William Hood, Chief Engineer of the Southern Pacific Co. Similar platform shelters have been used in the Southern Pacific station at San Francisco, Calif., and in the new union station at Denver, Colo. They were built by the Van Sant-Houghton Co., of San Francisco, Calif.

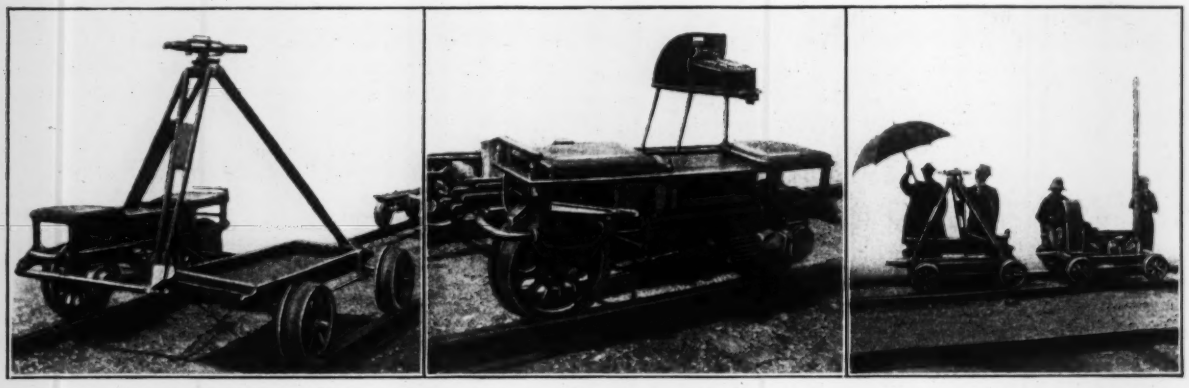
Artificial Ice Competes with Natural Ice—Although St. Paul, Minn., is surrounded by spring-fed lakes and has the finest natural ice in the world, an artificial ice plant turning out 30,000 tons per year has been found to be a paying enterprise during the first twelve months of its operation by the Citizens Fuel and Ice Co.

Field and Office

Innovations in Precise-Leveling Methods in Coast Survey

By William Bowie*

One of the most important changes in the United States Coast and Geodetic Survey precise-leveling methods is the mounting of the tripod on the frame of a railway motor car. John H. Peters, in charge of a Coast and Geodetic Survey precise-leveling party now operating in Indiana and Michigan, recently reported that he had leveled about 60 mi. with the tripod of his instrument mounted on a motor velocipede with which he transports his party. The results were so satisfactory that each of the three leveling parties of the survey now in the field will adopt this method of mounting the instrument.



FIGS. 1 TO 3. EQUIPMENT FOR FAST WORK IN PRECISE LEVELING, UNITED STATES COAST AND GEODETIC SURVEY
1—Level tripod mounted on railway motor car; 2—Adding machine by tabulating records; 3—Party and equipment complete

The 60 mi. were run in both forward and backward directions, and the accumulated discrepancies between the two runnings were only 3 mm. for the entire distance. The accumulated difference changed signs a number of times, showing that there was no systematic error in even a single running. It is certain therefore that the accuracy of the leveling is as great with the tripod mounted on the car as when it is set up on the ballast of the track, the universal custom to the present time.

The tripod is placed in position at the beginning of the day's work, and it is not dismounted until the party starts back to headquarters at the end of the day. It even remains on the car when the track is cleared for passing trains. Mr. Peters reported that the vibration of the car while traveling between instrument stations did not throw the instrument out of adjustment.

One can readily see advantages in this new way of setting up the tripod. It is free from the effect of the observer standing on the ties or ballast near a tripod leg. The car weighs 350 lb., so that the mounting is very firm and the height of the instrument remains constant while both sights are taken at a station. The time of setting up the tripod has been eliminated, and the time necessary to level the instrument is only a few seconds where the grade is uniform and the track straight. The leveling can be done quickly even where there are changes in grade and curvature of track between instrument stations.

It is possible that a mounting for the instrument different from that described may be devised which will be even better than the ordinary wooden tripod with the legs shortened. It might be somewhat more convenient to the observer, but probably would not result in greater speed and certainly not in increased accuracy.

There is no reason why the railroad leveling with the engineers' ordinary level should not be done in many cases with the tripod mounted on the car, as in the precise leveling by the Coast and Geodetic Survey. Even the hand velocipede car will no doubt be sufficiently rigid for the purpose.

*Chief, Division of Geodesy, United States Coast and Geodetic Survey.

Big Excavator Replaces Gold Dredge in Klondike

Nearly all the mining companies operating on a large scale in the Klondike on creek or river gravel use the ordinary type of floating dredge, but the North West Corporation, Ltd., is trying out the large electric-driven excavating machine illustrated in Fig. 1. This machine was reconstructed by Taylor & Hubbard, Leicester, England, from a German excavator[1], and was fitted by Leach & Goodall, of Leeds, with a washing plant and stacker for handling the coarse tailings. In the *Engineering and Mining Journal* of June 17, C. A. Thomas describes this excavator.

The complete machine weighs 170 tons and operates on two sets of tracks of 39½-in. gage, the outside rails being 23½ ft. apart. The machine is carried on four trucks having four wheels each. Each truck has two flanged

[1]Made by the Lübecker Maschinenbau Gesellschaft, Lübeck, Germany; described by George J. Young, "Brown-Coal Mining in Germany," Bull. Am. Inst. Min. E., February, 1916.

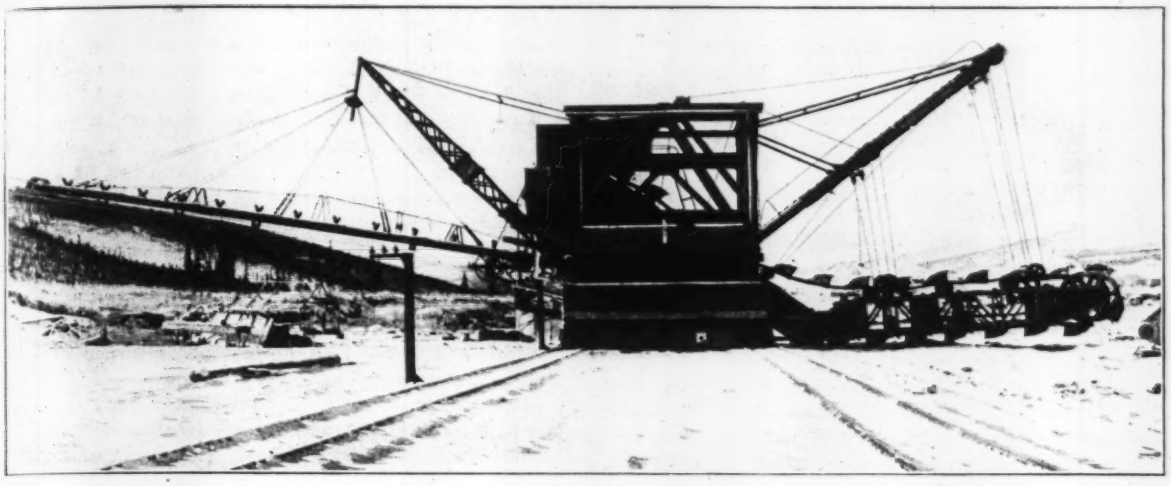


FIG. 1. LUBECKER EXCAVATOR REMODELED FOR GOLD DIGGING IN THE KLONDIKE

wheels on the outer side and two flat-surface wheels on the inner, the latter being the driving wheels to which power is transmitted by a set of bevel gears. The frame is composed principally of 12-in. channels and I-beams and is 27x22 ft. in plan. The excavator is 27 ft. 4 in. high.

The speed traveling parallel to the open cut can be varied from 3 to 10 ft. per min., and is determined by the depth to which the buckets are digging.

Buckets Fill in Traveling Toward Machine

The buckets are open-connected, of riveted sheet steel with a cast-steel cutting lip. They are connected by steel links and pins, a cast-steel lug riveted to the sides of the bucket forming one of the links of the chain. These lugs and connecting links traveling in a channel on the underside of the ladder keep the bucket down against the material to be excavated and take the entire stress of digging. The buckets are spaced 4 ft. 9 in. c. to c. The operation of the excavator is explained thus:

The buckets are filled as the line travels toward the machine on the lower side of the ladder, which is the reverse of the ordinary dredge operation. After passing over the intermediate tumbler, the full bucket line travels up a sheet-steel chute, being held about 3 in. from the bottom by cast lugs on the sides of the chute. At the same time that the buckets are being filled, the whole machine is propelled along its two sets of tracks, corresponding to the swing of the floating dredge.

As the line passes over the upper tumbler, the rear of the bucket disengaging from the connecting chain allows its contents to slide out the back end into the dump hopper. Before the bucket passes over the top of the upper tumbler it is in an upside-down position over the hopper. The empty line returns to the lower tumbler in suspension or carried on light flanged-wheel idlers.

As the gravel is dumped into the hopper, it is dragged to the starboard side of the machine by an endless raker chain composed of cast links and crossbars, supported and driven by cast sprocket wheels. At the end of the steel-plate compartment through which the raker chain carries the material to be washed there is an opening that allows all material to drop to a second compartment immediately below; and the same chain returning after passing over the sprocket idler, drags the material over a set of grizzly bars, approximately 29 ft. long. Two 8-in. spray pipes, extending the full length of this compartment, furnish water for washing and separating the fines and gold from the coarse material. This operation does away with the revolving or shaking screens of other types of placer-mining machines. The fines, after passing through the grizzly, are carried by steel sluices to the gold-saving tables. The coarse material is dragged by the crossbars of the raker chain past the grizzly-bar ends, where it falls into a metal chute and is conveyed by gravity to the stacker.

The tables are constructed in two 20-ft. sections of six sluices each, having a fall of 1½ ft. in 20 ft. and a total area of about 720 sq.ft. After the water and fines have passed over the last section, they are collected in a steel sluice and carried across the machine to the sand-elevator settling tank on the port side of the excavator. This settling tank is 27 ft. 7 in. by 54 in., with bottom sloping to the center, making a sump for the collection of sand for elevation by a bucket conveyor. These buckets elevate the sand to the top of the machine, where it is dumped into an extension of the steel chute that delivers the coarse material to the stacker. The water from the sand-elevator tank is conveyed across the machine to a series of steel settling tanks under the tables. From these tanks the water is repumped to the spray pipes over the grizzly bars, and in this way is used over and over for washing the gravel.

The stacker is 70 ft. long, equipped with a 24-in. canvas conveyor belt, and can neither be lowered nor elevated. The discharge end is 20 ft. above the rails.

The bucket line is driven by a 100-hp. motor through an intermediate shaft, on which is a friction clutch which can be set so the pinion will be automatically released if the stress becomes too great for the bucket line. A 30-hp. motor drives the ladder hoist, and it is this motor that propels the machine. The raker-chain, stacker, and sand-elevator drives are all taken from a countershaft connected to the 30-hp. motor. The electrical equipment was manufactured by the British Thompson-Houston Co., Ltd.

The machine is at present located on Dominion Creek, Yukon, Canada. Power will be furnished by the North Fork Power Co. from the North Fork of the Klondike River, 19 mi. from the present site. Current is transmitted at 33,000 volts and stepped down to 220.

The Stability of Material Towers of the temporary variety used to elevate building materials to the proper floor level, is an important consideration. This is especially true in view of the general spread of the compensation laws. In large cities towers of this character frequently serve a dozen stories and unless well-designed and constructed of sound timber add materially to the general hazard of the building trade. They must be braced and strutted and either guyed or anchored to the building. The subject is briefly treated in "Scaffolds," published by the Travelers Insurance Co., Hartford, Conn. This states that when a high building is to be erected it is better practice to add to the height of the hoist tower as demanded by the progress of the main construction. Sometimes a pair of towers are employed, in which case the two structures should be tied together by horizontal and diagonal stringers. It also notes that hoist towers are often sheathed with boards, and where this is done it is better to cover them on the inside. Doing this, results in a smoother surface for the elevator. This excellent recommendation is by no means general practice. The main objection is that the area of the hoistway is reduced, unless the plan dimensions of the tower are increased.

Tracks for Heavy Excavators on River Diversion

Unusually heavy portable-track construction is required to support the heavy excavating machines working on the river-diversion channel of the Little River drainage district, near Cape Girardeau, Mo. The machines are Bucyrus 175-ton and 200-ton dragline excavators, each carried by four 4-wheel trucks on two 3-ft. tracks.

The track construction is shown in Fig. 1. Each track is in sections about 18 ft. long overall and 8 ft. wide. The base of the section is a course of 4x12-in.

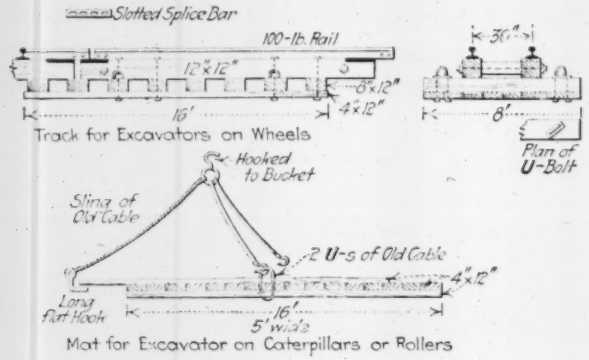


FIG. 1. TRACK AND MAT CONSTRUCTION FOR EXCAVATORS ON WHEELS AND CATERPILLARS

longitudinal planks, upon which are 8x12-in. crossties 8 ft. long, with two 12x12-in. track stringers carrying 100-lb. rails. The stringers are tied together by through-bolts and have transoms of spacing timbers between them. Under the ends of the rails are ½-in. steel plates 12x18 in., and ¾-in. bolts pass through the rail flanges and through the entire thickness of the track.

It will be seen that the track sections overlap. At one end the stringers project 30 in. beyond the base, and the rails project 10 in. beyond the stringers. Thus the rail ends and stringers of one section rest upon the stringers and crossties of the adjacent section. The rails are spliced with fishplates of 1x4-in. steel, with slotted holes.

The track sections are shifted as required by hitching them to the boom of the excavator. For attaching the slings there are four U-bolts, set diagonally in the ties. These are more satisfactory than eye-bolts.

The stripping of surface in advance of the main excavators is done by 90-ton Bucyrus dragline machines. Each of these is mounted on four caterpillars, as shown in Fig. 2; but even this base is not sufficient on the wet soft ground, which is further broken by numerous stump holes. To provide support for the caterpillar machines

FIG. 2. DRAGLINE EXCAVATOR MOUNTED ON FOUR CATERPILLARS ON PLANK MATS

and also for lighter machines that travel on rollers, heavy timber mats or platforms are used, as illustrated in Fig. 1. These are 18x5 ft., built up of three layers of 4x12-in. planks. The longitudinal bottom planks are 16 ft. long, and those of the top layer project about 30 in., so as to lap over the next section.

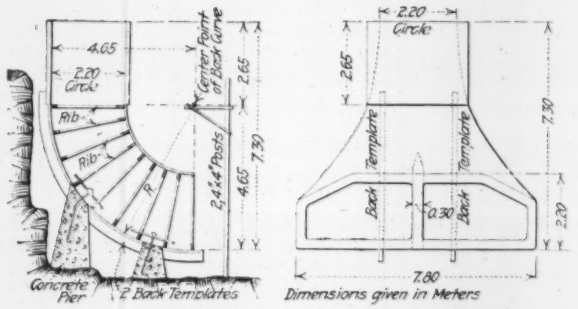
In shifting these sections they are attached to a three-part sling that is hooked over the excavator bucket. The sling is of old cable; one line carries a large flat hook to engage the projecting front edge of the mat; the others have ordinary hooks to engage U-loops placed slightly in advance of the center of gravity of the mat. The loops are made of old cable.

Information as to these details was furnished by S. T. Neely, Manager and Engineer of the Floesch Construction Co. (Rochester, N. Y., and Cape Girardeau, Mo.), which has the $1,260,000 contract for the work.

Forms for Heavy Concrete Draft-Tubes

By H. H. Braun*

In the construction of the five large draft-tubes for the 13,500-hp. turbines of the Ebro Irrigation and Power Co., near Lerida, Spain (described in *Engineering News*, Sept. 10, 1914, p. 536), the details of the formwork presented some difficulties. These tubes, of the general dimensions shown in the accompanying sketch, are fluted on the horizontal axis, turn a 90° curve and change from a circle at the top nearly to a rectangle at the bottom.



TIMBER FORMS FOR CONCRETE DRAFT TUBES

The forms were so large that it was necessary to erect them in place. Great care in location was imperative.

The top and bottom curves of the draft-tube are true circles, with the same center. The cross-section at nine different points, radial to the back curve, was determined and ribs built to these shapes, 1¼ in. being allowed all around for lagging. Two templets somewhat longer than the back curve and to the same radius, less 1¼ in. for lagging, were built of two layers of 3-in. plank and were made strong enough to bear the weight of the assembled form. One set of templets was used for all five draft-tubes.

Two posts were erected in front, bearing a light scaffold on which the true center of the back draft-tube curve was located with a transit and level. These centers were set about 3½ ft. each side of the center line of the unit, in line with the templets. The curved back templets were located by means of a 2x2-in. stick cut to the proper length, swinging from these centers, and then secured by blocking and bracing. The ribs were erected on these

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templets, properly spaced by measuring along the templet, and the direction determined by a string drawn from the centers on the scaffold. The ribs were held in place by bracing. The form was then lagged, with the exception of two 6-in. strips where the ribs rested on the templets.

As it was necessary to remove the blocking and templets before concreting, two small saddles, or piers, with six sleeve bolts in each, extending the entire width, were built underneath to carry the framing after the removal of the templets. The piers were concreted through holes in the draft-tube forms. After the concrete had set, the hold-down bolts were tightened, the piers stripped and the blocking and the templets removed, leaving the form securely bolted down on two concrete saddles. The mass concrete was then poured, no additional bracing being required.

The power system was built by the Pearson Engineering Corporation, under the direction of A. W. K. Billings, Managing Director, Fred W. Abbot, Manager of Construction, with the writer in immediate charge of power-plant construction. Alfred Sauerbraun was in charge of the Seros division.

Corner Inlet, Catchbasin and Curbing at Pueblo

Corner curb inlets and catchbasins are standard practice in Pueblo, Colo. Instead of being on a 90° arc the curb has two 45° arcs with a short tangent between. This allows the catchbasin inlet to come on a plane surface, as shown by Fig. 1.

The inlet is illustrated in Fig. 2. The interior is readily accessible from the gutter but not from the sidewalk. The top grating is designed to keep out leaves and other débris. Yet should the grating become clogged there is always the 4½-in. opening under the curb.

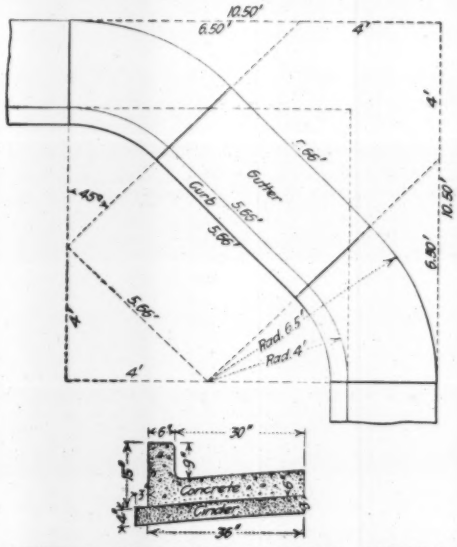


FIG. 1. DETAIL OF CORNER CURBING, PUEBLO, COLO.

The bottom of the inlet is built as a plane surface, sloping toward the sewer outlet. This is constructed of brick in the detail shown, but in practice it is customary to make this wedge-shaped masonry of concrete, using stone, brickbats and other coarse material. The inlet, and also all sewer manholes, are lined inside and outside

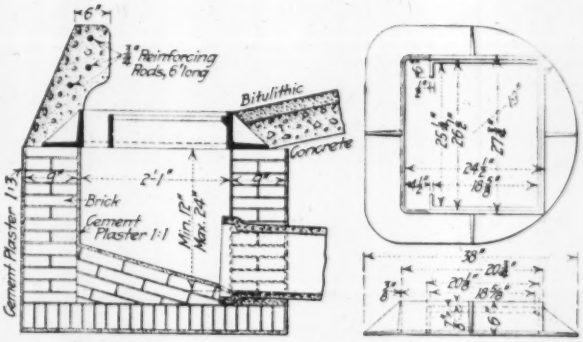


FIG. 2. DETAIL OF CURB INLETS, PUEBLO, COLO.

with a plaster coat of cement mortar, the mixture used inside being 1:1 and outside 1:3.

The foregoing data were obtained at the office of D. P. Gaymon, City Engineer, Pueblo, Colo.

Broken Cylinder Repaired by Autogenous Welding

By Patrick J. McGrath*

A steel pressure still for oil, 9 ft. 10½ in. in diameter by 30 ft. long, which had been hammer lap-welded in a gas fire originally, but had not been properly annealed, broke through the solid ⅝-in. steel plate for a length of 30 in. The break was parallel to, and about 12 in. from, one of the girth welds. At the time of the failure the cylinder was under a hydrostatic pressure of 150 lb. per sq.in. The crack was $^{3}/_{64}$ in. wide for practically its entire length.

Owing to the fact that it was not advisable to dismantle the fittings, it was necessary to make an autogenous weld. This was about 120° from the bottom (the cylinder being on its side) of the shell. It is generally conceded that the bottom of such welds contains slag and that a solid weld throughout the entire section is difficult to obtain in view of this fact.

*Summit Road and Park Ave., Elizabeth, N. J.

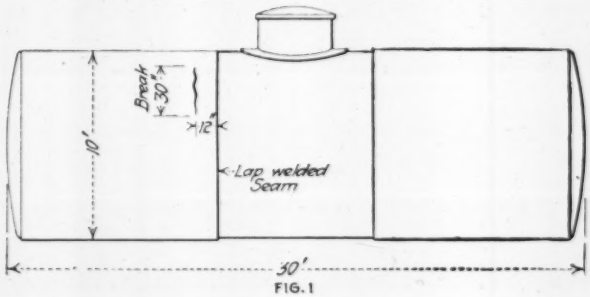


FIG. 1

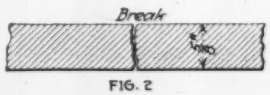


FIG. 2

FIG. 3

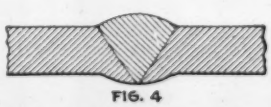


FIG. 4

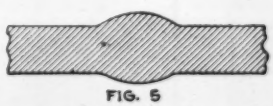


FIG. 5

FIGS. 1 TO 5. LOCATION OF BREAK IN CYLINDER AND SEVERAL STEPS IN WELDING IT

1—Location of crack. 2—Cross-section of plate, showing crack. 3—How crack was prepared for welding. 4—Weld completed but not dressed. 5—Finished job.

The break was prepared for welding by cutting the customary V to the full depth of the plate, after which the edges of the apex of the V were heated to red by means of the torch and, by using a hand fuller tool, were set down in the manner shown in Fig. 3. Before filling the V we welded crossbridges at 8-in. intervals to prevent drawing together by contraction of the welded parts. The after effects of contraction, due largely to heat that escapes into the body of the plate when welding, were minimized by pouring water on the plate about 8 in. from each side of the weld.

After welding about 4 in., we thoroughly hammered the weld and adjoining metal from inside and out alternately, meanwhile holding on with a sledge on the opposite side of the plate. After welding, the inner surface was chipped until the solid metal under the scale was reached, when it was finished by grinding out every vestige of the break with a portable emery wheel.

Dragline Wagon Loader for Roadwork

A combination portable dragline excavator and wagon loader has been devised by Barbeau & Werring, state road contractors, Minneapolis, Minn. One of these is now on gravel-road construction at Worthington, Minn.

A gasoline engine operates by chain drives two hoisting drums, mounted on a light steel-frame truck, as shown in Figs. 2 and 3. The hauling-line hoist is at the top of the triangular body frame, and the outhaul line is manipulated from a hoist below.

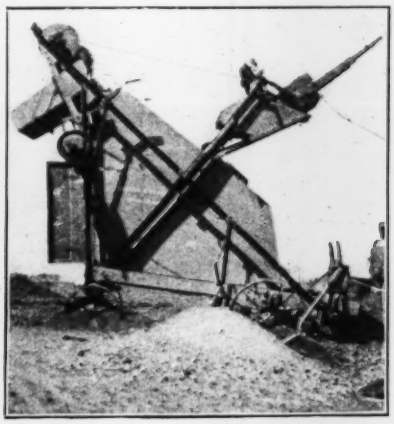
The loaded skip is drawn upon an elevating hopper which is raised into dumping position by the hauling line. At the top of the wagon frame the hopper and skip are in a position that causes the gravel to slide out through a chute into the wagon. No bin is provided.

The apparatus is still experimental and is doubtless capable of improvement. W. F. Rosenwald is Division Engineer for the State Highway Department on this work.

How to Handle a Misfire

When electric blasting caps are used and one or all of the holes fail, disconnect the wires from the blasting machine, and it is safe to go back immediately to investigate the trouble. The investigation should first consist of a search for broken wires, faulty connections or short-circuits.

When blasting caps and fuse are used, greater care must be exercised. It is never safe to go back immediately to a delayed shot that is primed with a blasting cap and fuse. The fuse may have been injured in tamping and instead of burning rapidly may smolder for a long time, then reignite the powder in the lower end of the fuse and fire the blast. The interval of waiting should be as long as possible, preferably until the next day if the firing is done in the afternoon. In all cases the wait should be several hours. When lighting fuse, be sure that the powder column is on fire, as time may be wasted in waiting for a blast whose fuse is not even lighted. Never use blasting-powder squibs and blasting powder for detonating dynamite charges.



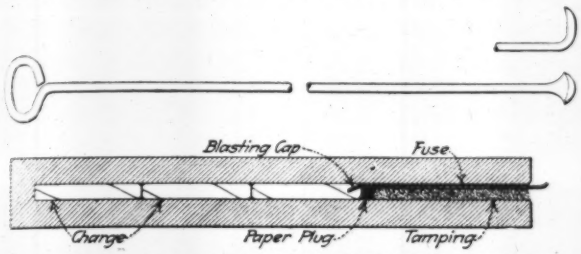
FIGS. 1 TO 3. A DRAGLINE WAGON LOADER ON MINNESOTA STATE ROAD CONSTRUCTION

If the hole is untamped or is tamped with water, make up another primer, place it on top of the charge, and fire. If the ground is soft and wet, put down another charge far enough away for drilling in safety, but close enough to cause detonation by concussion. When this is not possible, there are two methods of relief. The tamping can be carefully removed almost down to the charge with a spoon and hardwood probe. Extreme caution must be observed, especially when the cap is in the top cartridge of the charge, to prevent danger of firing the cap by friction or impact. The upper sketch (page 79) is a type of spoon recommended by the Du Pont Co.

When it is necessary to blast out a misfire by drilling and charging a second hole, much care should be used.

such a hole must be far enough away from the charge to make drilling and loading safe, but must be close enough to insure the old charge being blown out together with the rock or confining ground.

When, for any reason, misfires may be expected, a good practice is to put a wad of paper or dry leaves im-



UPPER—A GOOD SPOON FOR REMOVING TAMPING.
LOWER—PAPER PLUG PLACED OVER CHARGE

mediately over the charge of explosive and pack the tamping on top of this. If it becomes necessary to remove the tamping, this dry material makes an excellent index of just how far down it is safe to remove the tamping. All misfires should be placed under the direction of a careful and experienced workman, who should make his examination in a slow, methodical manner before beginning the work of repriming.

Where Dump Wagons Are Superior to Carts

Distributing concrete in drop-bottom dump wagons is unusual. For a haul of more than two blocks the wagon is superior on account of its greater capacity, in the same way that large wheeled-scrapers are superior to small slip-scrapers for handling earth where the haul is long. A longer haul is also said to be practicable, even up to 10 or 12 blocks, thus reducing the number of moves or set-ups of the concrete-mixing plant. With large loads the concrete does not set during transportation.

Further, where slow-setting cement is used, there is a possibility of establishing the mixer plant near the railway or source of supply (as with asphalt plants) and distributing all or a large proportion of the concrete from this one point.

One contractor who has used both one-horse carts and two-horse dump wagons is John A. Beebe, of Omaha, Neb. He states that as a result of his experience he sold the former and increased the number of the latter. He found it difficult to hire single horses; the cost was nearly as much as for a team, and the men leading or driving them were apt to walk slowly at the end of the day. Teams for the wagons were easily obtained; the load did not tire the horses, and as the driver rode, he did not slow down the pace. In this way from 50 to 60% more concrete was placed in a working day, while both men and horses were less tired after the work.

Another advantage is that the dump wagons can be used also for hauling dirt or other material, while the carts must stand idle when not in use for hauling concrete. His latest work was in paving at Benson, Neb. The haul was one to three blocks, using Stroud wagons with ¾-yd. loads. A little sand was thrown in the bottom to prevent leakage.

The Monarch Engineering Co., of Falls City, Neb., has used the same make of wagons for hauling concrete in street work at that city and at Sabetha, Kan. The haul was from two to four city blocks of 300 ft., and the load was 30 cu.ft. (two batches) of 1:3:6 concrete. This firm's experience with carts and wagons has been about the same as that noted.

At Aurora, Ill., a large amount of concrete has been placed in this way, especially the base for street paving. Several contractors have used Western dump wagons for hauling both excavated material and concrete. The same equipment serves for hauling gravel when weather does not permit of concreting. As to the possibility of leakage, Mr. Tarble, City Engineer, states that this can be avoided by using practically new tight-closing wagons and exercising proper precaution in cleaning out after each dump. There has been very little trouble of this kind.

Mark H. Lynch, of Boston, Mass., and the Eureka Construction Co., of Tulsa, Okla., have used Watson dump wagons in this way. Some dump-wagon builders state that their wagons have been used for this purpose, but cannot mention specific cases. With the use of the high-side dump wagon, special arrangements are required to get it under the spout of the mixer. Usually the driveway is excavated to some extent, and the mixer raised by placing it on planks or blocking.

Rock-Filled Wire-Mesh Bags for River Protection

The accompanying views show some of the flood-protection work on the Santa Ana River in California. The rock mounds or levees are laid inshore from the low-water lines of the river as a protection against encroachment in time of high water. The wire mesh is first laid in longitudinal strips and wired together. The boulders are

FIG. 1. ROCK MOUND LEVEES ON SANTA ANA RIVER

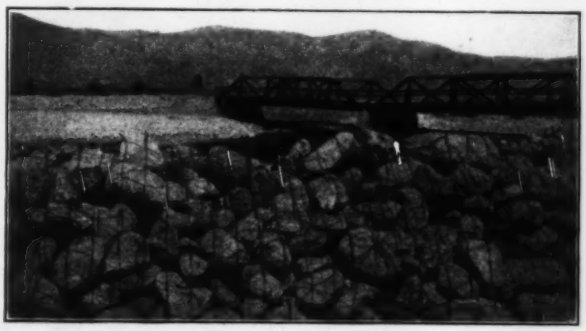
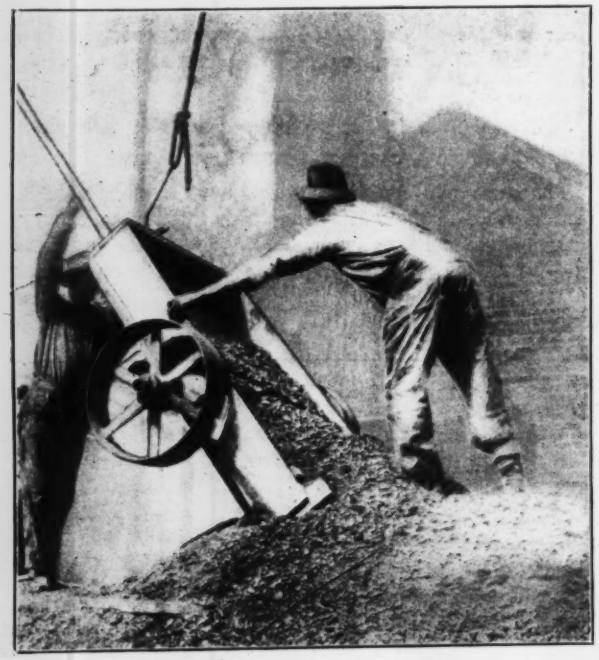


FIG. 2. NEAR VIEW OF ROCK MOUND IN WIRE MESH

then dumped on top to the desired shape and height, and the ends of the mesh are brought together and wired at the top. This forms a very flexible levee and one that will readily sink when undermined and thus stop undercutting. A similar system has been in vogue in India for some years. An installation there was described in *Engineering News*, May 6, 1909, p. 490. It is stated, however, that the system as used in California has been patented in this country by W. E. Pedley, of Riverside, Calif.

NOTES

A Gravel Cart Made of Scrap—The prohibitive cost of moving gravel by wheelbarrow up a steep grade was ingeniously overcome by a gravel cart built of scrap materials, by Edward Christman, well contractor, in building a well sump at Indian Head, Md., recently. The view shows the construction of the cart. Old pulleys on a piece of pipe formed the wheels and axle. The sills were made of 2x4s and the body was built of inch planks. A wire bridle served for towing, and the cart



HOMEMADE GRAVEL CART WITH PULLEY WHEELS; PULLED BY CABLE AND STEERED BY TONGUE

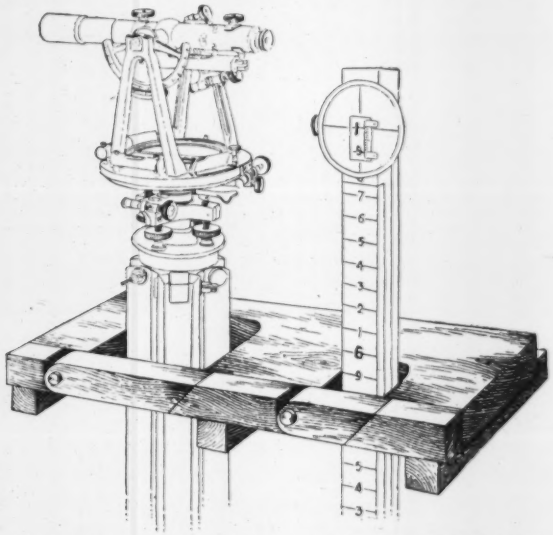
was steered by a 2x4-in. tongue. The motive power was the winch drum of the well-drilling machine, which wound in a wire cable running through snatchblocks and attached to the bridle. A load of 15 cu.ft. could be hauled 100 yd. and dumped, in a minute and a half.—Lieut. C. L. Lothrop, Jr., Indian Head, Md.

Office and Yard Lighting were discussed by Ward Harrison recently, at a meeting of the Association of Iron and Steel Electrical Engineers, in Pittsburgh. The following are his opinions: During the past year the tendency in office and drafting-room lighting, even in industrial plants, became so decidedly in favor of indirect or semi-indirect units that scarcely a factory building is now erected in which this form of illumination is not employed in the office space. In installing a semi-indirect system bowls of great density must be chosen, otherwise the results will be but little superior to those obtained from direct lighting. The density of the glass should be such that the bowls are not markedly brighter than the ceiling. The extensive use of gas-filled tungsten lamps in street lighting has led recently to the development of the prismatic glass reflector, in which the prisms are kept free from dust by being placed on the inner surfaces of two glass bowls which are then fastened together and made air-tight. This fixture is quite as effective in yard lighting as in street illumination. For a 300-watt gas-filled tungsten lamp the intensity at angles near the horizontal is nearly 900 cp., and it is the light at these angles that is most effective in illuminating distant areas and dark corners. Flood lighting is beginning to show a utilitarian value as well as one purely decorative. It is useful in furnishing light in and about large industrial plants. If a gantry crane, for example, is to be repaired at night and no convenient facilities are available for obtaining electric light close at hand, a projector may be mounted in a window of a building many yards away and the centre of operation as satisfactorily illuminated as if a portable cluster were used. Aside from the matter of night repair and construction work, it is worth while for the plant electrician to keep on hand a number of these projectors for operation in case of strike or other disturbances. Lighting up the approaches to a plant in this effective manner by a battery of units has obvious advantages.

An Artesian Well to Float Dredges for digging a drainage canal near Crawfordsville, Ark., is to be put down by the Canal Construction Co., of Memphis. Instead of waiting for the fall rain to fill the ditches, a centrifugal pump will be used to lift the water. Seven dredges will be used in digging the canal. The latter will drain 90,000 acres. The contract price for the canal is $47,000.

A Handle for Carrying Duct Tile is used by workmen of the Rochester (N. Y.) Gas and Electric Co. It is made by bending a stiff rod into S shape. The top is grasped by the hand, and the bottom is slipped into the tile. The vitrified clay has sharp and burred edges that cut the men's hands; formerly the tiles were carried by slipping a pick handle or stick through a duct.

Surveying-Instrument Rack for the Field Office—When we had an instrument on the job we always begrudged the time necessary to take the transit off the tripod, put it away in its box, etc. On a recent job of the Samuel Austin & Sons Co. a wall rack was made in which the instrument with its tripod could be placed and so the need for dismounting obviated. A small shelf, say 2 ft. long, was built about 4 ft. off the floor, high enough to come almost to the top of the tripod standing vertically. Two notches were cut in the shelf, one to receive the head of the tripod, the other to take rods. The



SURVEYING INSTRUMENT RACK FOR FIELD OFFICE

front of each notch was shut off by a vertically swinging gate which when closed locked in by the dovetail cut of its end. When the operator comes in off the job with the transit on the tripod he folds the legs of the tripod close together, stands it upright in its proper notch and shuts the gate in front of it. A possible improvement would be to provide a heavy oilcloth cover to put over the transit when it is in the rack.—Allen Brett, Cleveland, Ohio.

Removing Pulleys that have been rusted on shafts is frequently a troublesome job, but can generally be accomplished by heating the hub with a charcoal fire or some other means. The hub will expand, and the wheel can be easily removed. Care should be taken, however, not to heat the shaft, for if it expands as much as the hub nothing is gained.—"Power."

Editorials

Wanted: A Joint Committee on Nomenclature

The making of a technical definition is notoriously difficult. Local meanings are quite variable and the dictionaries are generally both inadequate and wrong. Consequently the individual or the small group that attempts to set down in precise terms just what some well known technical word is meant to convey is generally subjected to most critical comment, for it is strange but true that almost any engineer can on a moment's notice build up an opposition definition which he is prepared to defend against all comers.

And yet the fact is obvious that some standard definitions are most woefully needed. Take, for instance, concrete. It is safe to say that to nine engineers out of ten the word concrete—just plain unqualified concrete—means the hard stone-like substance resulting from the setting of a mixture of a hydraulic cement with some inert, inorganic aggregate. Yet the dictionary does not justify that definition, nor would the road men who talk of asphaltic concrete accept it. The narrow definition has been proposed by the Nomenclature committee of the American Concrete Institute. In contrast thereto, the Gypsum committee of the American Society for Testing Materials proposes a definition which would label as concrete a mixture of glue and sawdust.

This confusion is multiplied by the number of terms needing definition, particularly when such terms are jointly used in different industries. No small group can settle the difficulty. What is necessary is a joint committee of unquestioned standing and the widest of fields—a committee which will be able to weigh the claims for preference and will be of such authority that its pronouncements will by that fact *become* definitions.

What society will be the first to propose such an organization? It is no mean job that it would have before it; the French academy in many decades is only through the first dozen letters in its formulation of a dictionary of the French language. In its more restricted field a Joint Committee on Nomenclature would have just as hard and as useful a job.

Sensational Questions in New Column Tests

The real situation surrounding the new column tests of the Bureau of Standards appears to be that the results are very fair and satisfying on the surface, but deeper down give cause for serious concern. It is not possible to find the latter in the few scanty data released by the Bureau in its present short memorandum. Fortunately some additional information is available. The general figures of results justify the most optimistic news as to column strength. They show performances not approached by the bridge-builder's skill in prior construction. Earlier tests showed that modern columns of standard bridge steel will develop a high strength—practically the yield-point of the metal. The great Washington test series made by the Bureau for the American Society of Civil Engineers, fully discussed in these columns some months ago, may be referred to for clear proof of this fact. Now come the new tests with most gratifying confirmation of the general law, by showing that high-strength special steels will improve the resistance of the column in proportion to the increased strength of the metal.

Thus it now appears certain that special and alloy steels retain their full advantage when fabricated into columns. All the assumptions which lead to the choice of such steels for very large bridges are thus approved.

No less emphatic approval is given to the work of the designing engineer. In these tests he is found to have been completely successful in the endeavor to construct an integral strut, a column which will fail in the body and not in details. The engineer has applied the metal to column service with 100% efficiency.

Two-thirds of the struts in the test series failed by integral body curvature; every detail held up. Only one-third failed by local crippling, and even these came close to the full-efficiency mark, since their strength figures compare closely with those obtained with the body failures.

These two certificates of merit naturally produce feelings of satisfaction. But further we may congratulate ourselves on obtaining the valuable teachings of the several detail failures observed in the tests. There are disclosed certain slight weaknesses in latticing, in pin jaws, in thin-plate flanges stitched together. By improving these details we may reasonably hope to attain a state of full-efficiency designing for every column. We are nearly at this point of perfection now, but with the help of the tests we come measurably closer to it.

It is pertinent now to mention the fact that the last of these tests were made nearly two years ago. In spite of this lapse of time, which doubtless permitted the memorandum of results to be prepared with great care, we learn only the optimistic side of the case from the facts stated in the text and the table of results. The text omits all reference to certain observations of utmost weight in their bearing on column strength.

The simple fact is that while the ultimate static strengths of the columns tested are very high, elastic failure occurred at alarmingly low loads. This one fact overthrows the entire structure of optimism erected on the high ultimate-strength results.

Only one stress-strain curve is shown in the memorandum, and it is reproduced in our abstract. This shows what may perhaps be called fairly good elastic behavior. The elastic limit as marked is located well above the point reached by ordinary working stresses. True, it is only half the ultimate strength of the column, yet on the other hand no reason is shown for paying any more attention than is customary to this evanescent phenomenon of an elastic limit.

But, unrevealed by the memorandum of the Bureau, a whole series of stress-strain curves shows elastic limits

lying between 15,000 and 25,000 lb. per sq.in.! Does this mean that high-strength alloy steel is not even as good as ordinary bridge steel? Something must have been badly wrong with the material, or else the elastic limit is a snare and delusion and should be neglected in fixing safe working stresses.

The fact is, however, that these elastic limits were not mere irregularities in the stress-strain curve, but the points at which permanent set began. This was amply proved by release of load and reapplication. The point of beginning of set is universally recognized as an upper limit of possible loading, with regard to both variable static loads and to endurance. Therefore the truth appears to be that these columns of special steel, strong enough in the testing machine to go up to 50,000 lb. before failure, were not even as strong for service as columns of ordinary bridge steel with elastic limit of 25,000 lb.

A further fact of critical importance must be added: When, having applied such a load exceeding the elastic limit, the load is alternately raised and lowered a few thousand pounds per square inch, there is a continuous increase of permanent set. The column slowly yields under this jiggling of the load. The importance of this observation, even if only in a suggestive way, is too plain to be disregarded.

Precisely such load actions take place in the regular service of a column; the load is always varying, alternating from higher to lower. If in such service, with loads exceeding the original elastic limit, there is progressive yielding, then how can we depend on the ultimate safety, the endurance of columns subjected to such loads? It becomes clearer than ever, from this observation, that working stresses must be kept within the elastic limit, and that the special steel therefore failed to demonstrate its service value.

Part of the case is still without evidence, however. How does the column behave as to deflection while the increasing load is bending the stress-strain curve away from a straight line? Deflections are an important feature of column action, or rather of failure.

Observations show that there is no measurable deflection of the columns until a very high load is reached, a load well up toward ultimate strength. Very long before a load is reached at which deflection begins, the column has passed its elastic limit, and the diagram is curving decidedly as the metal undergoes permanent set.

This striking comparison, if it means anything, tells that the weakness of the column is not in its structure but in its metal. The engineer has done his part, and more than his part, in making the column keep in strict alignment up to a high load, even while the metal is compressing and undergoing permanent yield. The column as a structure holds up rigidly, with no trace of deflection, to a load-intensity of 40,000 lb.; but the metal in the column fails at half that load. The steel maker has not done his part.

These facts are so startling, and so new, that they surround the subject of steel columns with a wholly changed atmosphere. Most important deductions as to design and construction of columns are sure to flow from them, when they will have been confirmed broadly. It is conservative, therefore, to regard the publication of results of the new column tests as a sensational event in the field of structural knowledge.

Federal Funds Available for Road Construction

The passage by Congress of the act granting money from the Federal treasury to the several states for aid in road construction may be viewed from several different angles. There has been opposition to the bill on the ground that it was an attempt to make the wealthy cities and the populous states of the East, which contribute the bulk of the Federal revenue, help pay for the construction of roads in remote parts of the country. It has been predicted also that this bill is merely an entering wedge and that in the near future every rural congressman will go to Washington charged with the duty of bringing home as large appropriations as possible for the roads in his district, following the bad precedent set by the River and Harbor and the Public Buildings appropriation bills.

There is, however, a more hopeful aspect to this new departure in Federal legislation. That Federal aid to road construction was inevitable has been for some time recognized and was made clearly evident by the debate upon the bill in both houses of Congress. A large majority in both houses were strongly in favor of Federal aid and the debate was chiefly over the best manner of extending it. It should be understood that the Federal aid now extended is a very insignificant fraction of the total expenditure on road work in the United States. The $5,000,000 of Federal money which will be available next year is less than 2% of the amount which states and counties and towns are already expending on road work. If through the establishment of Federal aid and Federal supervision, an influence may be exerted which will cause the states to expend their own road appropriations with more intelligence and honesty, the indirect benefits resulting from the Federal appropriation may far exceed the amount of the money payment.

It is true that under the act as passed, Federal engineers are not to directly supervise the work of construction. The surveys, plans, specifications and estimates, however, are to be passed upon by the Federal authorities and the Secretary of Agriculture is not to pay over the Federal appropriation until he finds that the work is being constructed in compliance with the plans and specifications. Thus the primary responsibility for the conduct of the work is left where it should be left, with the state authorities which originate the plans; but it will be an excellent stimulus to good work to know that what is being done may have to undergo inspection by Federal engineers and that if the results of the inspection are unfavorable, the appropriation may be held up.

It is unfortunately the case that the great bulk of the highway and bridge construction in the United States is still carried on by county authorities without state supervision. County government is a matter in which the taxpayers and voters generally take comparatively little interest and as a result it is more or less permeated with politics and graft. Only a very small proportion of the counties in the United States employ engineers to carry on their construction work. Few of them in fact have work of sufficient magnitude to justify the employment of an engineer. There is no more important reform needed in the conduct of public work than the establishment of thorough state supervision of the work

carried on by counties, especially in the fields of highway and bridge construction and maintenance. It is to be hoped that the Federal bureau which is to be established to supervise road construction on which Federal aid is extended, will be able to wield a powerful influence toward bringing about this very desirable result.

Pumping Water for Power

Pumping water through a small head from one body to another to maintain an adequate supply through a considerable fall for the generation of power is not new. Especially has it been resorted to in emergencies. But so far as generally reported, the Bear Lake storage development of the Utah Power and Light Co., described elsewhere in this issue, presents the most extensive and interesting employment of this scheme. Bear Lake was an isolated body near the lower reaches of Bear River, but it was so advantageously situated that it could easily be made to receive the flood waters coming down from 3,000 sq.mi. of drainage area above, so as to provide regulation of river flow over any cycle of dry years on record. But the gradual paying out of the impounded waters was less simple. Obviously, there was the choice of a long and expensive outlet canal for gravity feed to the river or a short high-level canal with a large pumping station at the lake. Operating expenses of such a project were less than interest charges on the low-level canal plus the greater annual expense of maintaining the longer and deeper channel.

The pumping station itself is of great interest; looking at the section shown on another page, a striking similarity is seen with the latest type of hydro-electric generating station—witness the vertical-shaft pump with converging inlet passage, the spiral casing molded in the foundations, the direct-connected motor and the entire runner supported by a Kingsbury top bearing. Conveniently neglecting some details in the motors and impellers, this is a hydro-electric station running backward.

Accurate Cost Accounting for Government Industries

The Federal Trade Commission and the Interstate Commerce Commission, are insisting, very properly, on the establishment of accurate cost-accounting methods by the corporations under their supervision. This is justified on the ground that only through the facts furnished by an accurate and honest accounting system can information be obtained that will enable a regulating commission so to control the operations of public utilities as to deal justly with the public, the employees and the owners of the company's securities.

It would seem that the Government ought to do itself, in conducting its own business, what it compels the owners of a private business to do. Unfortunately, it is the rare exception when government accounts are so made up that the actual cost of a piece of work can be known. Not only this, but Congress seems very loath to apply to Government work the accurate accounting methods that it is prompt to demand in connection with private industry.

In the recent debate in the House of Representatives on the appropriation of $11,000,000 to establish a Government armor-making plant, Congressman Butler offered an amendment that expert chartered accountants should be employed to open a set of books which would show every item that was spent in connection with the construction of the Government plant and the final cost per ton of armor for each month of operation. The House, however, rejected this amendment.

There is without doubt a large body of public opinion which favors Government ownership and operation of plants producing war munitions, as well as public utilities. Those who hold such views, however, surely cannot consistently oppose the demand that records shall be kept which will show the actual financial results of Government operation. Refusal to permit the establishment of such a cost-account system is a strong indication that accurate records of cost are not wanted and that Congress very much prefers to have the records in connection with its Government enterprises in such a condition that they may be open to manipulation for political purposes.

Reputation Is Poor Bracing for a Roof

The belief that an architect's reputation or a building inspector's approval will hold up an unbraced structure received a rude shock from two recent cases of collapse, in Salt Lake City and Atlanta. Engineering science may note down in its records a double confirmation of the assumption hitherto made that what is not held up by solid substance of wood or steel or an equivalent is apt to fall down. Putting two and two together, *Engineering News* considers it as twice proved that a large roof requires, among other things, some tangible means of bracing in the longitudinal direction.

This demonstrated conclusion is recommended to the attention of the building-department officials of the above-named cities. By carefully heeding the teaching of the two accidents they may succeed in protecting the lives of their citizens in the future, though at the same time, of course, they will probably get into occasional controversy with some members of the architectural profession.

A circumstance well worth noting is that possibly these building officials could have secured for themselves the credit of deriving the important conclusion of the case, instead of leaving the discovery to *Engineering News*. It would have been necessary for them, however, to undertake the distasteful task of investigating the collapses. Concerning the Salt Lake City situation at least, the reporter who gathered the facts of the case found that city officials seemed desirous above all of forgetting the collapse and disclaiming responsibility. With this attitude of mind, of course, an investigation would have been unduly painful.

It is a pleasure to look back on the rapid and continuous progress made in city building-law administration during the last ten years. If the same rate of advance keeps up for the next decade, the building situation throughout the United States will be in excellent shape. It is certain that building inspectors will then be able to prevent the erection of unsafe architects' and carpenters' designs or, for that matter, of unsafe engineers' designs too. They will also have the power and the time, when accidents occur, to make prompt investigation, and utilize the results of the investigation to promote the safety of living in their city.

Letters to the Editor

Urging a Federal Department of Public Works

Sir—Referring to the editorial in *Engineering News* of June 8, "The Cohesive Power of Public Plunder," it is a healthy sign that the River and Harbor Bill appropriating $44,000,000 was carried in the Senate by a majority of only three votes. Comparing this vote with that on previous river and harbor bills, Senator Kenyon's prediction that "this is the last river and harbor bill framed along the lines of the present bill that will ever pass Congress" is likely to be fulfilled.

Is it not evident that a river and harbor bill to be of benefit to the country should be framed by men not only familiar with the needs of navigation, but with the special knowledge to say how those needs should be met?

The writer believes that the United States should have a Secretary of Public Works, whose sole and only business it should be to look after the public works carried on by the Federal Government, and whose honor and that of the President who appointed him would be either brightened or smirched according to the manner in which these works were carried out.

At present Federal public works are carried on by several departments, under subordinate bureaus. The Secretary of the Interior, the Secretary of War or any other secretary who has as part of his departmental organization some bureau that looks after some particular portion of the public works of our country need care little, so far as his standing as a cabinet officer goes, whether this particular public-works bureau is efficient or not. I cannot remember that any cabinet officer was ever severely criticized or that the President appointing such an officer lost any prestige because of wasteful or extravagant work in designing, building or maintaining any public work throughout the country.

The responsibility has been easily shifted. The buck has been passed with ease and celerity, excepting as criticism has fallen upon Congress at the time of passing the bill obtaining the funds for the work—and Congress is a hard thing upon which to fasten responsibility. Each individual composing it soon becomes an experienced buck-passer, and only in the past few years since sentiment has begun to crystallize somewhat upon these large appropriations has Congress felt any qualms upon the matter. I fully agree with you when you say:

> At least three-quarters of the senators fully understand the inherent corruption in the present method of making up river and harbor bills and would gladly welcome some method of framing such bills that would place the interests of commerce and navigation first and relieve members of Congress of the present responsibility for drawing all the money they can from the national treasury to be spent among their own constituents.

I sincerely hope that you will let no opportunity pass to recommend either the establishment of a Department of Public Works under a cabinet officer as secretary thereof, or some permanent national commission to control our public works for the benefit of navigation, as mentioned in the last paragraph of your editorial. You would be doing not only the engineering profession, for which you stand, but also the country at large a great service in pushing such a project. T. M. RIPLEY.

Watertown, N. Y., June 20, 1916.

Fruits of Parsimony

Sir—The only criticism of your recent editorial "The Fruits of Parsimony," is that the title is too narrow. The failure of the dam at Plattsburg was but typical of our national irresponsibility, of a pleasant freedom from worry about consequences. We are not in the least penurious, merely very confident that everything will come out all right. This confidence is most touching until it culminates in an Iroquois fire, a "General Slocum" disaster, or the burning up of a lot of girls in a shirtwaist factory. Then comes a wave of horror, which the optimist hopes works a tiny permanent advance, and the great stream of our self-centered interest rolls placidly on.

"Preparedness" is today a word to conjure with. May we not hope that with all the monster parades and loud acclaim we may be stirred to an advance in the Preparedness that embraces all other kinds and is fundamental—that against needless and foolish loss, whether of property, life or ideals? G. M. B.

Mr. Turner on Seattle Test

Sir—I note regarding your discussion of the flat-slab floor test at Seattle that you have not been quite as ready to accept a test on partly cured work as some other engineering papers. This test is merely a repetition of past history. In the John Deere Plow Co. building a $50,000 bond was written by me guaranteeing deflection on cured work. A premature test was made of a panel with a resulting deflection of 2 in. which should not have exceeded $\frac{5}{8}$ in. When the work was thoroughly dried out and cured the deflection was precisely that figured; to wit, $\frac{7}{16}$ in.

The relative deportment of the wall and interior panel is another indication of the condition of the work. The deflection of a wall panel is twice as great as an interior panel before the concrete is thoroughly dried through at the neutral plane. Afterward it is substantially the same. This phenomenon is familiar to all builders and is readily accounted for.

The displacement of the steel noted may account for 20% difference in deflection when the work is thoroughly cured; in the writer's judgment not more than that.

The results of merely making a test on partly cured work are much less serious, but involve the same error.

The erroneous idea that the number of days that concrete has been cast determines whether it is cured or not is still believed in to some extent by many engaged in the concrete business who have had experience in putting up two or three buildings in warm weather. The idea that concrete is safe when it is cured on the

top and bottom surface of the floor and not cured in the center has resulted in the erroneous belief honestly entertained by many that there was no relation between deflection and stress in reinforced floors. No relations exist when the concrete at the neutral plane is not thoroughly dry and hard. When it is thoroughly cured so that there is no appreciable horizontal shearing deformation at the neutral plane, there is a precise relation between deflection and stress. This relation between horizontal shearing rigidity and flange rigidity when understood will account clearly to all for such freak results as that of the Seattle test. When the promised supplemental test is made, the accuracy of this statement will be apparent.

In the case of the John Deere test made eight years ago, there were many engineers who honestly entertained the view that 50-day concrete was as hard as the concrete ever would be, and if it would not carry the load with the guaranteed deflection at that time it never would. They could not conceive that the dampness of the basement and lack of circulation of the air could have caused the drying out and hardening to be slow.

C. A. P. TURNER.

Minneapolis, Minn., June 26, 1916.

Many Unbiased Flat-Slab Tests

Sir—That part of Mr. Godfrey's letter (published in *Engineering News*, June 29, 1916, under the heading "Pittsburgh Flat-Slab Ruling and the Seattle Test") which contains his claim that the Seattle "mushroom" building "satisfies Mr. Ferguson's ruling exactly" has been answered by Mr. Ferguson by definite calculations, showing the unwarranted nature of the claim. My own figures, made some weeks ago, showed practically the same result—namely, that the stresses observed in the Seattle test are what would be expected under the Pittsburgh ruling. The floor tested does not approximate a Pittsburgh design in either strength or stiffness.

But Mr. Godfrey's letter contains other statements equally unwarranted, which should also be briefly noted in your journal for the sake of accuracy. Mr. Godfrey refers to the Seattle test as "the solitary disinterested test," the plain implication being that all previous flat-slab tests are dishonest and unreliable. At my invitation Mr. Godfrey witnessed the test of the Larkin warehouse in Chicago in 1912 and saw there Professors Talbot and Slater and Messrs. Abrams, Nelson, Gonnerman and others from the University of Illinois Engineering Experiment Station at work. The readings were taken by these men and the test conducted with their expert knowledge and experience throughout. The test of the Franks Building in Chicago was conducted by Prof. W. K. Hatt, of Purdue University, assisted by Professors Slater and Skofield and others. Several other tests have been conducted under the same scientific auspices, and for my part I do not see how any fair-minded engineer can make such statements as Mr. Godfrey does.

In the Larkin test three adjacent wall panels were loaded before Mr. Godfrey's eyes; in other tests, accounts of which have been generally published, wall panels have been loaded, and yet Mr. Godfrey has repeatedly protested in print that wall panels are never loaded. In one test the wall-panel load was held in place for a full year. Of course, I understand that these tests afford no support for Mr. Godfrey in his present position, but it is only fair to point out that this letter of June 29 is based on a deliberate disregard of the great accumulation of scientific test data previously made by many agencies and on erroneous and unwarranted conclusions from the data of the single test actually considered.

ARTHUR R. LORD.

Chicago, Ill., July 3, 1916.

Twin-Pipe Problem Solved by Hydraulic Slide-Rule

Sir—Mr. Kingsbury's solution, in *Engineering News* of June 29, of the flow in a double pipe line with varying sizes and with several coefficients by the Williams-Hazen formula is interesting and correct. It may be useful to your readers, however, to know that there is a much easier solution for this common problem. It can be carried out with the hydraulic tables mentioned or, still better, with the hydraulic slide-rule by the aid of which the tables were made.

Assume a quantity of water, say 10 million gallons, flowing through one of the lines, say the 30-in. line used by Mr. Kingsbury. With the coefficients assumed by him, the loss of head in this line for this quantity is found to be 1.67 ft. The same loss of head divided by the length of the other line gives a slope for it of 1.11 per 1,000. With this slope the second line is found to carry 4.83 million gallons per day. The total discharge is the sum of these quantities, 14.83 million gallons per day, with a loss of head of 1.67 ft. Putting the marker on the hydraulic slide-rule over a slope of 1.67 and bringing over it the figure 14.83 on scale No. 2 (taken to represent flow in gallons daily), the marker may then be moved and will give directly the quantities corresponding to any other heads. For instance, with 8 million gallons per day the head is found, by a single movement, to be 0.53 ft., as stated by Mr. Kingsbury. The proportions of flow through the two lines will be as the amounts found in the first computation—namely, as 4.83 and 10 are to the total of 14.83.

The results obtained are identical with Mr. Kingsbury's. The calculation is more rapid and no diagram need be made. The extension of this principle allows rapid solutions of still more complex problems involving multiple lines and lines in each of which there may be several pipe sizes involved.

ALLEN HAZEN.

30 West 42nd St., New York City, June 30, 1916.

Water-Supply Preparedness

Sir—The need for "preparedness" in water-supply matters is illustrated by the reported unpreparedness at Camp Whitman, brought out by the sudden call for mobilization of the New York State troops. If such conditions can obtain at a camp where preparations for reception of troops must have been under way for some time, it is not to be wondered at that emergency encampments are often so illy served.

To meet such situations, does it not seem desirable that systematic investigations should be made of the resources of the country for supplying safe water and maintaining supply systems under war conditions? The need of this seems to apply both to present reserved or appropriated areas and to locations that might later be needed for troop camps, hospitals, detention and concentration purposes.

For such a purpose clear and concise data should be available regarding present sources in order to judge of their ability to deliver adequate emergency supplies; the possibilities of additional undeveloped sources; the location of suitable materials to construct temporary supply works; and the availability of local water-works departments to supply men and materials for emergency work. Based on this information, general plans could be formulated as to what to do and how to do it quickly and efficiently, should necessity arise.

As the health and comfort of both soldiers and civilians depend largely on safe and adequate supplies of water, it seems desirable to take early measures to meet such emergencies as can be foreseen. This duty should probably come under War Department control, and most fittingly it seems now might be assigned to the Medical Corps. That division of the army is particularly concerned with the preservation of health and therefore seems best equipped to administer this work.

The details of organization and method, however, in order to get best results, should be directly under the supervision of competent water-works men familiar with both construction and maintenance problems. In the past many data have been collected for Government bureaus by well-meaning but untrained men and have failed to reach their utmost possible value.

In the unlooked-for event of an invasion of this country or the bombardment of its seacoast cities it is possible that the work suggested might also be of great value in helping to maintain present supplies and preventing or remedying serious interruption of service from enemy activity.

CALEB MILLS SAVILLE.

Hartford, Conn., June 26, 1916.

Cross Cut Power Station

Sir—The Cross Cut power plant described by J. W. Swaren in your issue of Apr. 20, 1916, was, before its completion, considered by some a questionable innovation in the development of low-head water power, and the engineers responsible for its design were subjected to much unfavorable criticism. As pointed out by Mr. Swaren, the silt-laden water, the necessity of fully utilizing the varying stream flow without interfering with irrigation demands below the plant, the long pipe lines and the extreme temperature variations each contributed to the difficulties of the problem. Each of these conditions has been successfully met by the design of the plant, and the merit of the design has been demonstrated by the efficiency attained and the satisfactory operation of the apparatus.

Reaction turbines operated on the Salt River project, with practically the same character of water at heads of about 200 ft., deteriorated so rapidly that their wicket gates and runners required renewal every three or four years, yet the Cross Cut wheels after about a year's operation still show an efficiency of over 79%, or 4% above the guaranty.

The wear has been confined almost entirely to the jet deflectors, and even these parts are kept out of action to a great extent by using a number of nozzles just sufficient to carry the maximum load. It is a rather significant fact that the manufacturers consented to an efficiency test to determine a bonus or penalty of $1,200 for each per cent. after the wheel had been in more or less continuous service for over a year. Practically full-load efficiency maintained down to one-sixth power by successively cu ting out nozzles.

The plant is being maintained and operated in thr 8-hr. shifts by five men, including the chief operator a a janitor, and this is the best answer to any question to the satisfactory operation of the equipment. A num ber of unexpected conditions developed when the pla was put in operation for the first time, and the enginee of the Pelton Co. are entitled to credit for their persev ance and ingenuity in correcting these difficulties. Th fundamental conception of the plant and in fact many o the details of design are due to O. H. Ensign, Consultin Engineer for the United States Reclamation Service.

JAMES M. GAYLORD,
Electrical Engineer, United States Reclamation Service.

Denver, Colo., June 15, 1916.

Railway Curves Measured on Arc or Chords

Sir—Responding to the note by T. A. Palmer in *Engineering News* of June 29, p. 1240, may I suggest that some years ago Professor Philbrick issued a railway engineers' fieldbook in which the definition of the degree was based on the arc of 100 ft. instead of on the chord. A very few years ago in response to a suggestion of mine made to the Track Committee of the American Railway Engineering Association the question whether to use this method or to continue the use of chord-degree curve was debated somewhat and put to a vote. There was an overwhelming majority in favor of present practice, but a respectable minority in favor of the system suggested by Mr. Palmer. At page 18 of the text on "Railroad Field Geometry," by the writer, the arc method is recommended; but, because of the action of the railway association, it was not generally used in the book, nor was it used (for the same reason) in the "Railroad Engineers' Field Manual," although on page iv of the preface the method is again recommended.

By the way, the reviewers of this same field manual (which is based on the decimal division of the degree instead of the sexagesimal) have almost unanimously indorsed the system, but with practically equal unanimity have said that American engineers are too conservative to introduce it, which strikes one as curious.

WILLIAM G. RAYMOND.

State University of Iowa, Iowa City, July 1, 1916.

Sir—Regarding the inquiry of T. A. Palmer, there are many answers in the files of *Engineering News;* a fair sample is on p. 17 of the Engineering Literature Supplement, Feb. 16, 1911. As in all such discussions the old experienced engineer has the last word. But he is wrong, even though trains have gotten around his curves these many years.

We stick to the old approximate method only from superconservatism; for the same reason that radically different track and rolling stock are used here than in England under exactly the same conditions.

Mr. Palmer is on the right track for peace of mind and a clear conscience but he should not try to convert any one.

A. M. HAYNES.

Denver, Colo., July 5, 1916.

New Facts About Materials and Methods of Test

A considerable number of important facts on properties of materials and tests, and descriptions of several new forms of test apparatus were brought out in technical papers read before the American Society for Testing Materials at its annual meeting, Atlantic City, N. J., June 27 to 30, 1916. The essential points are abstracted very briefly below, together with additional information or comment in discussion at the meeting.

The papers of most immediate practical bearing, perhaps, are those on hardening and on grain growth in steel, and on the cracking disease of brasses. They, as well as other parts of the proceedings of the meeting, put strong emphasis on micrographic-structure study, and a side-issue consequence of this is that the society will endeavor to standardize micrographic magnifications.

Cracking of Brass Tubes and Its Cure

The puzzling defects of rolled and drawn brass and bronze articles, which caused great loss of money on the Catskill Aqueduct, have produced corresponding trouble and expense elsewhere. Brass condenser tubes have given particular trouble from splitting. The Detroit Edison Co. could get no promise of relief from the condenser manufacturers. A. E. White then investigated the matter for them. He found the cause of splitting to lie in slip-planes developed in the grains by cold-working and allowed by insufficient annealing to persist. A specification was developed, based on the Navy Department specification, with micrographic tests added. The microscope is to detect unremoved strains in the metal (slip-planes) and also show whether the grain size is coarse; in both respects it is a check on annealing.

Very closely defined temperatures of anneal are vital. The change in structure was investigated with the microscope, and it was found that no change of structure from the original cold-worked and presumably injured condition was produced until an annealing temperature of nearly 400° C. was used. With annealing temperatures of 600° and over, the size of crystal grew very large, especially with long-continued heating. This represents damaged constitution, which must be avoided. The microscope clauses of the specification, based on these facts, are inserted with a view to insuring anneal on the one hand at sufficiently high temperature to efface the mechanical-strain injuries and on the other hand low enough to avoid coarsening of grain.

The tubes in question were of 60% copper and 40% zinc, approximately. The chemical impurities were of small amount, not enough to be a probable cause of the splitting.

Strong comments tending to uphold present rolling-mill practice were offered by W. R. Webster and W. B. Price. Mr. Webster, for instance, stated that since the navy's Bureau of Steam Engineering 15 years ago adopted a specification covering final annealing of brass condenser tubes, millions of tubes have been made without a single failure reported. Ductility and tube service have not been shown to have any connection. Claims that the degree of drawing per pass should be limited to avoid injury have not been demonstrated. There is no experience to show that annealing between passes must be at limited temperature to avoid grain growth. Mr. Price went farther than this, saying that the claims for light drafts are contrary to fact. At a reduction per pass of 30%, or three numbers on the B. & S. gage, crystal change occurs, and such heavy drafts are therefore necessary.

C. M. Chapman referred to the past difficulty of getting good condenser tubes and inquired for a reason. Mr. White, on the amount of reduction per pass, conceded that further investigation of the point is needed. In support of his specification he stated the simple fact that up to July, 1913, the company had found no relief from "season cracking" of condenser tubes, but thereafter, with the new specification, complete relief was obtained.

Saw-Toothed Stress-Strain Curves of Bronze Wire

Tests of bronze and similar wire by A. V. De Forest showed jagged or saw-toothed stress-strain curves under certain conditions. Only the part of the curve above the elastic limit was thus affected. Phosphor-bronze wire showed this effect slightly when heated to 126° C., and strongly at 180° C. Some brass wire showed it at room temperature when the load was applied slowly. A very effective special testing apparatus was used for the work, in which load was applied by water, and both the stretch of the wire and the load were registered by the motion of a beam of light on photographic paper. The author offered an explanation of the jagged character of the curves, based on the hypothesis that there is an amorphous substance in a thin layer between the grains of the metal and the further assumption that this substance becomes viscous at certain temperatures and allows a limited slip.

Prof. H. M. Howe, mentioning the fact that similar saw-toothing has been observed in stress-strain diagrams of steel, set forth some difficulties in the way of the author's explanation of the phenomenon.

Strong Aluminum Bronze

Alternating-stress tests of aluminum bronze showed very satisfactory endurance, as reported by W. M. Corse and G. F. Comstock. Comparisons were made with manganese bronze (of higher static strength), and the aluminum bronze stood a larger number of alternations. Both the Landgraf-Turner machine, which tests with large degrees of bending, and the White-Souther machine, which applies a definite stress (usually within the yield limit), were used.

Heat-treatment of aluminum bronze was found to produce higher elastic properties without greatly reduced ductility. The pieces were quenched in water from 850° C., later reheated to 650° C. and allowed to cool in the furnace. This treatment gave a proportional limit of 40,000 to 56,000 lb. per sq.in., a tensile strength of over 90,000, and elongation and reduction of area of 12 to 18%. The alternating-stress endurance also was good.

Hardness Changes in Hardening

In hardening steel at various quenching temperatures, Prof. H. M. Howe and A. G. Levy discovered striking discrepancies between hardness and transformation of structural constituents of the steel. Lower quenching temperature (but always above recalescence) produced little change in hardness, while the density increased progressively, showing progressive transformation of the hard to the soft constituents. Still lower quenching temperatures produce, first, a sudden loss of hardness at the recalescence point, and then a continued slower decrease of hardness, while the structure change is slight. The authors refer the change of hardness below recalescence to coalescence of the cementite and ferrite, regarded as a deterioration.

These investigations, made on 0.92 carbon steel, involved the use of both the Brinell ball method and the Shore scleroscope method of measuring hardness. The latter method was found to be specially valuable because it tested small areas. Certain observations on peculiarities of the Shore test are included in the paper ("Notes on the Hardening and Tempering of Eutectoid Carbon Steel and on the Shore Test").

Water-Quenched Steel Axle Equal to Oil-Quenched

Very thorough tests on two driver axles were made by the Pennsylvania R.R. to learn whether oil quenching or water quenching in the hardening process gives better results. The differences found were slight, but generally in favor of water quenching.

The two axles, 10 in. in diameter, forged from the same heat of steel, contained about 0.55% carbon, 0.57% manganese and 0.20% silicon. They were quenched at 1,550° F., one vertically in water and the other horizontally in a heavy oil, and then were heated to 1,175° and 1,200° F. respectively and cooled in air. From each, 45 longitudinal test specimens were cut, at five different places along the axle, in three radial planes and at three different distances from the inner bore. The grand average of all 45 pieces gave:

Quenching	Elastic Limit, Lb. per Sq.In.	Ultimate Strength, Lb. per Sq.In.	Elongation in 2 In., %	Reduction of Area, %
Water	54,509	96,482	23.1	42.2
Oil	51,118	96,415	24.2	43.4
Minimum of A. S. T. M. specifications	50,000	80,000	20.0	40.0

Micrographs showed good sorbitic structure, with diffuse ferrite in the water-quenched and a fine network of ferrite in the oil-quenched axle. Chemical analysis showed 10 to 15% segregation close to the inner bore, which is unusually high.

These results, reported by C. D. Young, Engineer of Tests of the railway, were interpreted by him as showing an advantage, or at least no disadvantage, in the use of water as quenching medium. He referred also to long experience of one treating plant with water quenching as showing that there is no undue danger (of cracking) in the use of water when the work is done carefully.

Nearly every phase of the steel subject is related to the question raised, as the discussion indicated. Quenching results are determined by rapidity of cooling, but not by the nature of the medium (Prof. H. M. Howe; Lawford H. Fry). The internal stresses existing in the metal during the quenching probably affect the transformation of structure in the axle (Prof. H. M. Howe). The oil-quenched axle may be the tougher on account of its lower elastic ratio. Tests have shown that while, before drawing back, oil-quenched steel has lower elastic limit than water-quenched, the change produced by drawing back is not the same for the same reheating temperature, and with increasing reheating temperature the elastic limit of oil-quenched steel holds up better and finally passes that of water-quenched steel (R. P. DeVries).

As to drawing-back temperature, Professor Howe said:

> It should exceed 400° C. by only enough to reduce the quenching stresses to harmlessness, or rather to reduce them till the benefit from their further removal ceases to equal the harm done by the accompanying increase of coalescence. If, for instance, the quenching stresses become thus harmless on drawing at, say 500°, then 500° is the proper drawing temperature. Should this leave the strength too great and the ductility too small, it would seem better to get the needed weakening and ductilizing through smaller carbon content than through higher drawing temperature.

But, replied Mr. Young, the treatment used is known from practice to give good service results. Whether lower ductility would still give good service is not known, but practical railway men would not like to try it. Lower drawing temperature would not meet the ductility specification. On the other hand, lower carbon would make it impossible to get the elastic limit up to 50,000. The treatment used gives the ductility of full annealing and 50% higher elastic limit (C. D. Young).

Cracking during hardening is not due to segregation, since case-hardened pieces do not crack and since the axles reported did not crack, although they had high segregation. The differences of the two sets of results in the paper are very slight as compared with the differences of methods of procedure, so that probably the latter are more responsible than the quenching medium (J. S. Unger).

A fundamental question was put by Prof. H. M. Howe: Is the objection to high elastic limit in railway steel a superstition, in view of the good service given by hard-drawn wire?

Forced circulation of the cooling medium was reported by L. H. Fry to give surprising results. In a comparative trial using compressed air as a means of producing circulation the elastic limit was 69,000 and the ultimate 105,000, while without circulation the figures were 49,500 and 95,000; the ductility was practically the same for both pieces, 21% elongation and 42% reduction of area. Details of the test were not given. He expressed the view that all cooling media produce cracks and that the best results are obtained by using oil and subsequently applying a proof test.

Dangerous Ferrite Growth in Low-Carbon Boiler Tubes

In forcing its boilers at high rates to carry peak loads, the Detroit Edison Co. had trouble from frequent bagging. The forcing is such that melted ash is found on top of tubes over 30 ft. above the grate. There is some scale formation, although only distilled water is used for make-up. Bagging occurs under scale deposits. A. E. White and H. F. Wood believed that the bagging was due, at least partly, to deterioration of the tube steel from re-

crystallization of the ferrite to a coarse structure. In sections of the tube wall they did indeed find coarse structure in a thin layer at the fire side of the wall. This led them to make studies of the conditions under which grain growth of low-carbon steel takes place.

Grain growth results from heating, but only after cold working or deformation below recalescence. The boiler tubes (seamless drawn) being deformed at low temperature, their heating in service might cause grain growth. Many careful experiments were therefore made on 0.11% carbon-steel samples, by annealing thoroughly, making a deep Brinell-ball impression on one face, cutting apart through the depression, and studying the structure by microscope before and after prolonged heating. Different heating temperatures required different lengths of time to produce grain growth. A law connecting the required time in minutes with the temperature in Centigrade degrees was deduced:

$$Temperature = 722° - 52.4\ log_{10}\ time$$

The experiments covered a temperature range from 550 to 675° C. It is planned to try higher-carbon tubes, of about 0.35 to 0.40% carbon, to cope with this coarsening of structure.

Sauveur, in 1912, showed that only in a fixed range of deformation does grain growth of ferrite take place. Jeffries recently showed that for every temperature there is a critical strain, and for every strain a temperature, producing grain growth. Pearlite, however, is a bar to growth of the ferrite when it incloses the latter. Sauveur never found growth with steel over 0.12% carbon; and if this proves to hold true, it would not be necessary to raise the carbon of tubes very much in order to stop the coarsening (H. M. Howe).

The weakening effect of recrystallization extends through only a small part of the tube section, for which reason it is hardly a sufficient explanation of bagging. The great drop in the tensile strength of steel at high temperatures—to practically zero above 800°—is enough to account for the action. If prior overstrain were responsible for recrystallization, the latter ought to be found inside the metal rather than at the outer surface. High-carbon steel will show the same kind of drop in strength at high temperatures as low-carbon steel, and the tubes will be harder to set tight and keep tight. The solution is to hold down the forcing of boilers. The tests were made on steel just under the line of 0.12% carbon. There is no difficulty in making or applying drawn tubes of 0.15 to 0.18% carbon (F. N. Speller).

The tubes in question showed recrystallization for a depth of one-eighth the thickness in from the fire side. At every point of bagging there was found either coarse crystallization or blowholes. Experience shows no difficulty in rolling 0.35 carbon tubes in, or in keeping them tight (A. E. White).

Proportional Limit Identical with Elastic Limit

That there is no good ground for distinguishing proportional limit from elastic limit was asserted by several expert testers with wide knowledge of materials, in a topical discussion. J. E. Howard, in taking the view that sets do not develop before the proportional limit, objected to the term yield-point as lacking definite meaning. His discussion directed special attention, however, to the factors which give rise to early irregularities in the stress-strain diagram, including local sets due to initial strains or to nonhomogeneity and imperfect manipulation. He asserted that preliminary straining is not a proper means for removing initial strains and that annealing is the only satisfactory normalizing method.

T. D. Lynch advocated fixing the yield-point at an elongation of 0.5%; for steels below 100,000 lb. per sq.in. yield-point this should agree well with the value read either with dividers or by drop of beam. He drew a distinction between material characterized by a slow curvature of the diagram between proportional limit and yield-point, and material characterized by abrupt change (the two limits close together). The abrupt type of curve may be produced in material of the gradual class by mechanical work or by heat-treatment. The difficulty of determining yield-point by dividers or drop of beam in material of the gradual class was claimed to justify the proposed definition by an elongation figure. Mr. Lynch holds that in modern work designers must have accurate information on the proportional limit.

H. F. Moore and F. B. Seely concluded from test data and experience that probably no stress-strain curve is straight in the lower part and that curvature starts at once. If this is true, the proportional limit is a matter of the sensitiveness of the instruments used. Some precise test measurements were made to get a comparison of various methods proposed for elastic-limit determination. In these tests an averaging extensometer was found to give a diagram equivalent to the average curve constructed from gage readings on three lines. The influence of the grips—threaded and wedge—was found to be erratic, and more work on the subject is needed.

R. P. De Vries cited elastic hysteresis as proof that speed of straining does affect the stress readings. The existence of a yield-point as a practically useful limit must be recognized. H. L. Morse advocated dropping the term elastic limit in favor of proportional limit, the two defining the same point. L. H. Fry shared in the latter view. C. D. Young called attention to the new definition of elastic limit just adopted by the Steel Committee as one practically useful in the laboratory that must accept material quickly and determine the critical points without tedious refinement. This definition simply makes change of rate of elongation the criterion and leaves open the question of how it is to be observed; an extensometer with dial gage is used for the purpose at Altoona. Uniform rate of applying stress is premised.

Measuring Linseed-Oil Foots

In view of special attention directed during the past year to the possible harmful influence of foots, or deposits, in linseed oil, interest attaches to the method devised by C. D. Holley for quick separation of foots in the laboratory, determining their amount with reliability and distinguishing an oil that has been duly aged from one that has not been aged. The method depends on treatment of the oil with phosphoric acid and naphtha, after treatment allowing the liquid to separate, when foots collect on top of the phosphoric acid. H. A. Gardner suggested heating and letting the oil stand over night, to precipitate albuminous substances, after which the benzine thinning will cause the foots to settle out quickly.

[*Further abstracts will appear in coming issues.*]

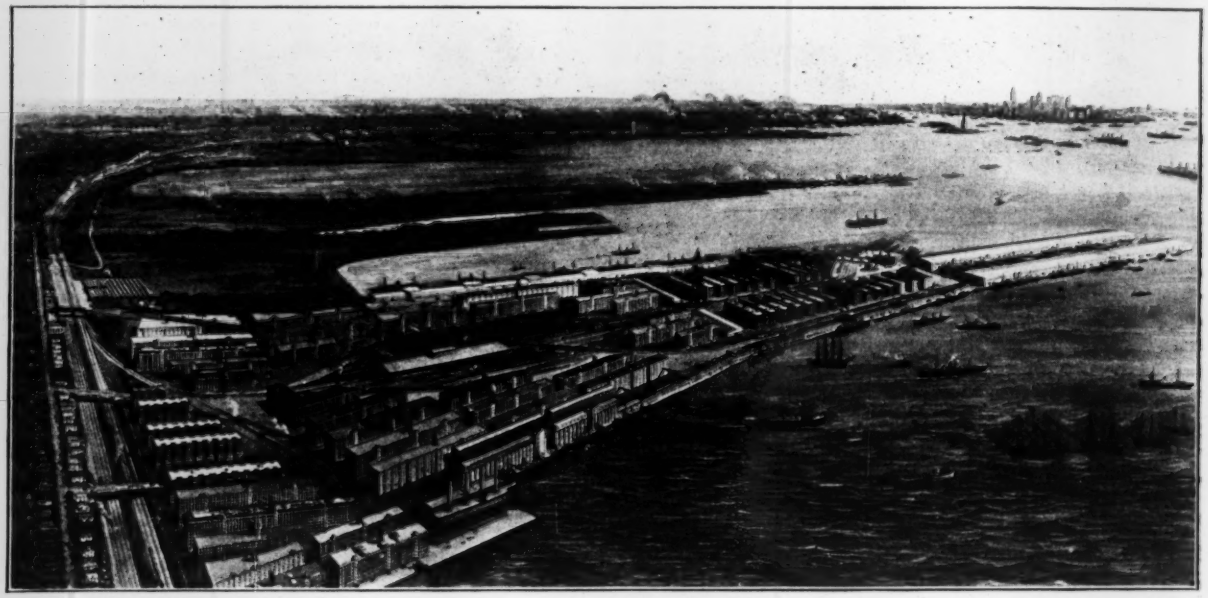
New Jersey Freight Terminal for the Port of New York

Every engineer who has studied the problem of freight handling in the port of New York understands that for economic transfer between ocean steamships and railways leading to the West a great freight terminal should be created on the New Jersey side of the North River.

The port of New York is the gateway for nearly half the foreign commerce of the United States. A very large proportion of this commerce does not originate at New York, but is brought there by the railways leading to the West. The ocean steamships entering the port are docked on the waterfront of the Boroughs of Manhattan and Brooklyn for the most part; and the few which are docked on the New Jersey side of the river have no railway lines reaching their piers. Thus on practically all the foreign commerce handled through the port of New York there is involved the cost of lighterage between the railway terminal and the steamship, and a vast amount of trucking through the streets as well. The lighterage alone costs probably on the average not less than $1 a ton, and the cost of trucking is an exceedingly heavy burden upon New York commerce. If, however, the railway car could be brought alongside the steamship, at a terminal on the western side of New York Harbor, all this expense would be eliminated.

During the past year the New Jersey Board of Commerce and Navigation has been actively studying the commercial development of the New Jersey shore of the North River. Plans have been developed by the Chief Engineer of the board, B. F. Cresson, Jr., for a great steamship and railway terminal to be located on the waterfront of Bayonne, between the Greenville terminal of the Pennsylvania R.R. on the north and the Standard Oil refineries and docks on the south. Negotiations are now in progress between Irving T. Bush, President of the Bush Terminal Co., and the city authorities of Bayonne, having in view the construction of a great freight terminal on this site, similar in general to the Bush plant in Brooklyn. As at present projected, the city and the corporation to be organized by Mr. Bush would jointly furnish the capital for the creation of the terminal and would share in the control of the property and in the profits.

The accompanying perspective sketch has been prepared from the preliminary plans to indicate something of what such a great terminal plant might eventually become. The ultimate development shown in the view would cover an area of about 400 acres of land now under water. The total volume of fill required to be placed to raise the now submerged ground to an elevation of 10 ft. above high water is estimated at 5,000,000 cu.yd. Much of the material for this fill would be obtained from the dredging necessary to make the channels. The plans contemplate channels of a depth of 35 ft., 350 ft. in width on each side of the peninsula and extending inward from the bulkhead line a distance of about 4,700 ft. The two piers that extend from the bulkhead line outward to the pier-head line are each 225 ft. in width and 2,400 ft. long, with a waterway between that is 290 ft. in width. The view shows these piers covered with one-story sheds. On the bulkhead inland from the piers a row of multistory warehouses for freight in transit is provided. Located at the center of this artificial peninsula is a railroad yard with a capacity of 2,000 to 2,500 cars, and tracks lead from this yard out onto the long piers and to every building on the property. From this yard direct rail connections can be made to all the railways terminating on the west shore of the Hudson. The plans also include grain elevators, a marine coaling plant, cold-storage warehouse, cargo-handling cranes and warehouses and factory buildings.



RAILWAY AND STEAMSHIP TERMINAL PROJECTED BY CITY OF BAYONNE AND IRVING T. BUSH

At present, plans are being worked out to determine with what minimum investment an operative plant can be created sufficiently large to provide the essential features of economic operation and capable of development with the growth of business. From present indications it is probable that such an initial plant can be constructed in Bayonne at a cost of $8,000,000 to $10,000,000.

Final Report on Elmhurst Disposal Plant

A final report to the Board of Estimate and Apportionment, City of New York, on the Landreth electrolytic sewage-disposal plant at Elmhurst, Long Island, has just been made by the City Board of Consulting Engineers (made up of the Chief Engineers of the Board of Estimate and Apportionment and the consulting engineers of the several boroughs of the city. The board considers the continued use of this plant as too expensive and uncertain and recommends the temporary use of chemical precipitation and sludge-drying beds at this point pending a permanent connection to projected sewers.

History of the Elmhurst Plant

In 1900 a disposal plant was authorized by the Board of Public Improvements for Elmhurst. It consisted of four settling tanks and three sand filters. On account of the continuance of objectionable odors, in 1914 a small (25,000 gal.) experimental Landreth plant (employing simultaneous lime precipitation and electrolysis) was installed (by the Electro-Chemico Corporation, of Philadelphia). Reports on this machine were made (1) by P. M. Travis, Chemist, and E. I. Firth, Engineer, for the Borough of Queens, (2) by R. H. Brown and W. T. Carpenter for the Bureau of Sewer Plan, (3) by F. A. Cleveland for the Bureau of Municipal Research, and (4) by E. J. Fort and C. E. Gregory for the Bureau of Sewer Plan.

A large unit, capable of handling 750,000 gal. per day was installed in 1915 and two studies have been made on that, (5) one by W. P. Mason, J. C. Olsen, C. O. Mailloux and P. M. Travis for the Electro-Chemico Corporation, and the other (6) by the Bureau of Sewer Plan for the Board of Estimate and Apportionment.

There was not entire accord in the results of these investigations. For instance the first (Travis and Firth) report showed a requirement of 144 kw.-hr. per 1,000,000 gal. and 1,257 lb. lime. This electrolytic machine (vertical type) was 24x18 in. in section and 7 ft. 3 in. high. The current was split between carbon and steel electrodes, and paddles were revolved between these plates. No great technical skill was required for operation and no apparent wear was found on the iron electrodes. Immediate coagulation, wonderfully quick settlement and bactericidal action (99.819%) beyond possible action of lime alone was reported. The dissolved oxygen was "considerably increased" and the oxygen consumed by organic matter "enormously reduced."

The later reports (3) were less optimistic. For instance in the Brown-Carpenter-Cleveland series, time of passage through the machine was given as 2¼ min. and the time of filling the sedimentation tank 1½ hr. The previous high bacterial reductions were not repeated (only from 30 to 94% in effluent from the machine and from 97 to 100% in the settlement-tank effluent). The machine was found very sensitive to change in character of sewage. The reduction in total suspended solids was between 65 and 83%. The reduction in suspended organic matter was 3 to 7% in the machine effluent and 38% in the tank effluent. The reduction in consumed oxygen ran from 2 to 95% for machine effluent and 23 to 80% for tank effluent. The increase in dissolved oxygen for these two effluents was 18 to 87% and 53 to 68.5% respectively.

On running the machine with lime only (without current) the chemical results and bacterial reduction in machine effluent were equal to that of regular operation—except in the appearance of the flock and the higher dissolved oxygen content after sedimentation. Without the paddles the flock was formed slower and the bacterial reduction less. No bacterial reduction in machine effluent was obtained in operating with lime added at 375 lb. and 684 lb. per 1,000,000 gal. On increasing the content to 1,040 lb. the reduction was 72.3%. On 2,210 lb. the reduction was 94.1%. About 17½ cu.yd. of sludge (94% moisture) was formed per 1,000,000 gal.

The large plant as tested by Messrs. Mason, Olsen and Mailloux employed in order (1) screening, (2) addition of lime, (3) electrolysis, (4) sedimentation, (4) sludge pressing. The machine was a covered horizontal box 3x2¾ ft. in section and 23¼ ft. long. The flow was between sets of parallel steel plates with paddles revolving parallel to the plates. The maximum values of current supply were 125 volts and 40 amp. The time of flow through the tank was 68 sec. The time in the sedimentation tanks was about 4 hr. With 1,000 to 1,300 lb. hydrated lime per 1,000,000 gal. the raw sewage and machine and tank effluents showed, for instance, oxygen saturations of 25.5%, 31.8% and 45.5% and bacterial counts of 1,100,000, 846,000 and 800.

When lime was used in the machine (1,510 lb. per 1,000,000 gal.) but no current, the machine effluent became darker with smaller flocks. The bacterial reduction in raw sewage and machine and tank effluent was from 210,000 to 11,150 and 61,000 (for the last, bacteria rose to 9,360,000 after standing two days). The percentage of saturation for dissolved oxygen rose from 28.4% to 43.9 and 44.3%; the consumed oxygen in the final effluent was 25.7%.

In regard to costs, for a 1,000,000-gal. plant, Messrs. Mason, Olsen and Mailloux reported: 1,300 lb. lime, $5.14; current, 85 kw.-hr. for electrolysis, 60 kw.-hr. for agitation, 174 kw.-hr. for pumping, and 16 kw.-hr. for lime vat, at 4c., $13.40; sludge pressing and removal $5.11; labor and supervision $12.50; maintenance and supplies $1; total $37.15.

On account of the lack of accord of the reports they were all submitted to Earl B. Phelps of the United States Public Health Service Hygienic Laboratory for review and report. His findings were (1) that the final effluent was highly satisfactory, (2) that the method of sludge disposal was not satisfactory and could not be employed where there was real necessity for sewerage purification, (3) that there was good basis for honest difference of opinion as to whether the electrolysis contributed materially to the results, (4) that the cost was too great to compete under usual conditions with better known processes.

In considering what to recommend at Elmhurst the City Board of Consulting Engineers considered the annual cost per 1,000,000 gal. of the following: Electrochemical ($48.19) and activated sludge ($39.38) processes, sprinkling filters ($36.45), chemical precipitation ($30.63), pumping to 51st St. sewer ($24.05) and to Booth St. sewer ($22.15). The electrochemical process was dismissed because of cost and the uncertainty of results with stronger sewage.

News of the Engineering World

Reconstruction of Notable Large Bridges

The reconstruction of two noted bridges over the Ohio River will be a feature of the bridge engineering work of 1916. The first of these is the bridge of the Pennsylvania Lines at Louisville, Ky., which was built in 1869 and has therefore seen 47 years of active service. It was designed by and built under the direction of the late Albert Fink, then Chief Engineer of the Louisville Bridge Co. It consisted almost entirely of deck-truss spans of the Fink type. The two channel spans had subdivided triangular Warren through trusses, and the swing span had trusses with ordinary triangular web system.

Beginning at the north (Indiana) end the bridge had one span of 100 ft., one of 149 ft., three of 100 ft., 400-ft. channel span, six spans of 245½ ft., 370-ft. channel span, two of 227 ft., two of 210 ft., two of 180 ft., four of 149½ ft., 264-ft. swing span over the Louisville & Portland Canal, and two 50-ft. spans to the Kentucky approach. Interesting observations as to the longitudinal creeping of the bridge spans were given in *Engineering News*, Mar. 28, 1885.

There may be some changes in the span lengths of the new bridge. The War Department required a clear span of at least 200 ft. for the new draw, as noted in *Engineering News* of Dec. 18, 1913, and the railway company had proposed a cable-operated lift bridge over the canal in place of the present 264-ft. swing span. The company is now in litigation with the Federal Government in regard to the latter's requirements as to the spans, and until the decision is rendered by the United States Supreme Court no definite action can be taken. The design and construction of the new bridge will be under the direction of J. C. Bland, Engineer of Bridges for the Pennsylvania Lines. A contract has been placed for part of the work, and preparations are being made for the removal of the first spans at the south end. For this reason the local train service over the bridge has been discontinued.

The second Ohio River bridge referred to is that of the Cincinnati Southern Ry. (Cincinnati, New Orleans & Texas Pacific Ry.), at Cincinnati, Ohio. This present structure was completed in 1877. The main bridge has two 300-ft. truss spans at the north end, a 519-ft. channel span, a 370-ft. swing span and a 110-ft. shore span at the Kentucky end. This main structure was built by the Keystone Bridge Co., under the direction of its designer, J. H. Linville. The north or Cincinnati approach has six truss spans of 121 to 191 ft., and was built by the Baltimore Bridge Co. The structure is single-track throughout, with rails 106 ft. above low-water (or about 35 ft. above high-water).

The present piers will be used, with minor alterations at their tops, but will carry a double-track superstructure. The new bridge is being designed by Ralph Modjeski, consulting engineer, of Chicago; M. B. Case, who has been resident engineer for Mr. Modjeski on the new Memphis bridge, is now in charge of the surveys for the new Cincinnati bridge. The work will be under the general direction of Curtis Dougherty, Chief Engineer of the Cincinnati, New Orleans & Texas Pacific Ry. (Queen & Crescent Route).

Two other examples of the reconstruction of the superstructures of notable old bridges may be mentioned. The first is the Mississippi River bridge at Keokuk, Iowa. This was designed by J. H. Linville and built by the Keystone Bridge Co. in 1869-70. Its reconstruction (on the old piers) was completed early this year, and was described in *Engineering News* of Apr. 13, 1916. The second is the Missouri River bridge of the Union Pacific R.R. at Omaha, Neb. This was built in 1872 and partially rebuilt in 1887. The reconstruction was planned under the direction of R. L. Huntley, Chief Engineer of the Union Pacific R.R., and work is now in progress.

National Engineer Reserve Act Now Effective

The legislation necessary to create a national engineer reserve was recently passed by Congress and signed by the president, becoming effective July 1. This bill, known as the Army Reorganization Act of 1916, provides for the organization of an Officers Reserve Corps including an engineer section or an Enlisted Reserve Corps. The War Department is now formulating the plan of organization and the requirements that will be prescribed for enrollment. As soon as practicable the War Department will invite the engineers of the country to apply for commissions in the reserve corps of engineers, and there will be available for those interested full and complete information as to the requirements for commissions in the several grades, from second lieutenant to major, and for master engineers and sergeants, and the method of procedure to secure commissions or enlistment. The following is an abstract of the principal clauses relating to the Officers Reserve and the Enlisted Reserve Corps.

The commissions of all officers of the Officers Reserve Corps shall be in force for a period of five years unless sooner terminated in the discretion of the President. Such officers may be recommissioned, either in the same or higher grades, for successive periods of five years, subject to such examinations and qualifications as the President may prescribe and to the age limits prescribed herein: Provided, That officers of the Officers Reserve Corps shall have rank therein in the various sections of said Reserve Corps according to grades and to length of service in their grades.

In time of actual or threatened hostilities the President may order officers of the Officers Reserve Corps, subject to such subsequent physical examinations as he may prescribe, to temporary duty with the Regular Army in grades thereof which cannot, for the time being, be filled by promotion, or as officers in volunteer or other organizations that may be authorized by law, or as officers at recruit rendezvous and depots, or on such other duty as the President may prescribe. While such reserve officers are on such service they shall, by virtue of their commissions as reserve officers, exercise command appropriate to their grade and rank in the organizations to which they may be assigned, and shall be entitled to the pay and allowances of the corresponding grades in the Regular Army, with increase of pay for length of active

ervice, as allowed by law for officers of the Regular Army, rom the date upon which they shall be required by the terms f their orders to obey the same: Provided, That officers so rdered to active service shall take temporary rank among themselves and in their grades in the organizations to which ssigned, according to the dates of orders placing them on ctive service; and they may be promoted, in accordance with such rank, to vacancies in volunteer organizations or to temporary vacancies in the Regular Army thereafter occurring in the organizations in which they shall be serving. . . .

To the extent provided for from time to time by appropriations for this specific purpose, the Secretary of War is authorized to order reserve officers to duty with troops or at field exercises, or for instruction, for periods not to exceed 15 days in any one calendar year, and while so serving such officers shall receive the pay and allowances of their respective grades in the Regular Army: Provided, That, with the consent of the reserve officers concerned, and within the limit of funds available for the purpose, such periods of duty may be extended for reserve officers as the Secretary of War may direct. . . .

For the purpose of securing an additional reserve of enlisted men for military service with the Engineer . . . Corps . . . of the Regular Army, an Enlisted Reserve Corps, to consist of such number of enlisted men of such grade or grades as may be designated by the Presicent from time to time, is hereby authorized, such authorization to be effective on and after the first day of July, 1916.

Limiting the Flow Through the Chicago Drainage Canals

A Senate amendment to the River and Harbor Appropriation Bill limiting the diversion of water from Lake Michigan through the Chicago drainage canals to 250,000 ft. per min. was noted in *Engineering News* of June 8, 1916, p. 1108. The Conference Committee eliminated this amendment in its report submitted on July 3 and at the same time struck out the provision granting authority to the State of Illinois to expend $5,000,000 on the improvement of the Illinois River in accordance with a law passed by the Illinois Legislature in June, 1915. Unless some change is made in the bill prior to its final passage by both houses, which is very unlikely, Congress will therefore have taken no action either legalizing or limiting the Chicago drainage canal and the State of Illinois will lack the authority to proceed with the work of improving navigation on the Illinois River.

Structural Steel Companies in New Combination

The merger of the Pennsylvania Steel Co., Steelton, Penn., and the Maryland Steel Co., Sparrows Point, Md., announced some time ago, has been completed by the formation of a new company known as the Penn-Mary Steel Co., South Bethlehem, Penn., which will be a subsidiary of the Bethlehem Steel Co. A deed transferring the property of the Pennsylvania Steel Co. to the Penn-Mary company was filed at Harrisburg, July 7. It called for a consideration of $10,167,297. A mortgage held by the Bankers' Trust Co., Trustees, of New York City, for $60,000,000 was filed against the property at the same time.

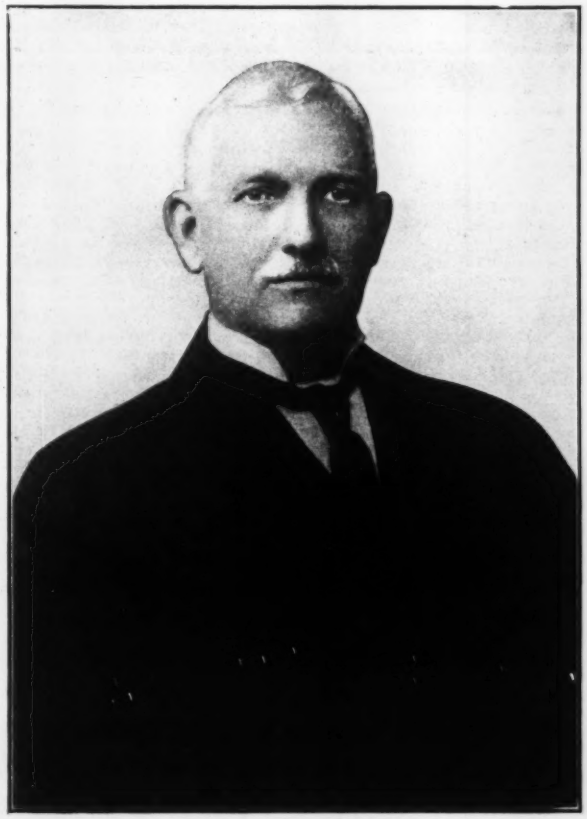
Changes in the engineering and executive departments of the Steelton plant of the Pennsylvania Steel Co. have been announced by Vice-President W. F. Roberts, who is in charge of all plant operations, as follows: President, E. C. Felton, retires automatically; and while no place has yet been designated for Frank Tenney, secretary, he will be retained in some capacity. Frank B. Carney, superintendent of the Steelton plant, is made general metallurgist for the Bethlehem, Steelton and Sparrows Point plants with headquarters at South Bethlehem. William S. Rutherford is made head of the order department of the entire company with headquarters at South Bethlehem. Thomas Earl, superintendent of the bridge and construction dpartment of the Steelton plant; Charles H. Mercer, chief engineer, and Lewis E. Johnson, assistant to Superintendent Earl, are transferred to South Bethlehem, where they become part of the sales and engineering force in charge of designing and field erection in connection with structural work. Carl B. Ely, assistant to Mr. Earl, has been appointed superintendent of the bridge and construction department in charge of operations.

Joseph Otis Osgood

Joseph O. Osgood, chief engineer of the Central R.R. of New Jersey, whose death on June 28 was noted in last week's issue, was born at Cohasset, Mass., in 1848. He took special courses at the Massachusetts Institute of Technology in 1872 and 1877, but had long before then engaged in engineering work.

At 17 years of age he joined the engineering corps of the Eastern Shore R.R. (now a part of the Pennsylvania) as a rodman. Subsequently he was assistant engineer, and from 1870 to 1871 was transitman and resident engineer on the location and construction of the Portland & Ogdensburg Ry. He was made assistant to the chief engineer of the Maine Central R.R. in 1871.

Following his studies at the Massachusetts Institute of Technology, Mr. Osgood was made resident engineer



JOSEPH OTIS OSGOOD

for the State of Massachusetts on the improvement of the South Boston Flats. In 1878 he went West and for two years was division engineer of the Atchison, Topeka & Santa Fe Ry. on construction work in Colorado and New Mexico, and for two years was chief engineer of the California Southern R.R.

From 1883 to 1884 he was chief engineer of the Boston, Hoosac Tunnel & Western R.R.; after a year in private practice he became chief engineer of the Toledo, St. Louis & Kansas City R.R., and in 1887, chief engineer of the Lake Shore & Michigan Southern Ry. For the 12 years following 1889 he was a consulting engineer with offices in New York City. Since 1901 he had been chief engineer of the Central R.R. of New Jersey.

The Excavation for the Rogers Pass Tunnel was finished on July 6. In the past year more than 3½ mi., has been excavated—working from two headings. This double-track tunnel is being built for the Canadian Pacific Ry., by Foley Bros., Welch and Stewart. A. C. Dennis is superintendent for the contractor.

Tribulations of Boston Public-Works Engineers — Since Mayor Curley removed 17 employees of the Public Works Department of Boston last January by abolishing their positions he has put nine of them back into other and subordinate positions. These do not include the three who resisted the mayor's action by legal means, and who, though ordered reinstated by the court, have not yet been put back. The case of these three is still under consideration by the city law department, which is planning to have the decision reviewed by the supreme court of the state.

The American International Terminal Co. has been incorporated in Delaware as a subsidiary of the American International Corporation which was organized some months ago by interests connected with the National City Bank of New York. The new corporation is organized to deal with the problem of transportation terminals, warehouses, etc., and will begin its work with a study of the conditions in New York City. The Vice-Presidents of the company are Geo. J. Baldwin and W. S. Kies; among the Directors are a number of officers of the National City Bank and E. F. Webster, Henry R. Hays and O. E. Stevens of the Stone & Webster Co. William H. Lyford, of Chicago, has been appointed Consulting Engineer of the company.

Railway Employees Engaged in Train Service throughout the United States are now taking a general ballot giving authority to the officers of the trainmen's brotherhoods to declare a strike in case the railways will not grant the 8-hr. day with time and a half for overtime which the brotherhoods have demanded. Congress has as yet taken no action on the request of the National Chamber of Commerce that the Interstate Commerce Commission should be given authority to make an immediate investigation. At the recent convention of the Brotherhood of Locomotive Firemen and Engineers in Denver, a resolution was adopted protesting against any action by Congress. In preparation for the possible emergency of a strike of its employees engaged in train service, the Pennsylvania Railroad Co. issued a circular to its employees calling for volunteers to fill vacancies in the train service in the event of a strike and announcing that if such volunteers are called on for duty they will be given permanently the positions to which they may be assigned. Many of the employees now engaged in train service are receiving two or three times the wages of equally competent men employed in the repair shops, and are working shorter hours. The Pennsylvania announces that between 50,000 and 60,000 of its employees have volunteered in response to this circular, or more than twice the number that would be needed to take the place of the engineers, conductors, and other trainmen in the event of a strike.

The Federal Aid to Road Construction Bill, which was signed by the President on July 11, is substantially the Senate bill with a half-dozen minor changes. The bill makes an appropriation of $5,000,000 for the fiscal year ending June 30, 1917, and this appropriation is increased $5,000,000 for each successive year until for the year 1921 the sum of $25,000,000 is appropriated. Control of the appropriation is placed with the Department of Agriculture, which is authorized to deduct not exceeding 3% of the appropriation for the expenses of administration. Distribution of the appropriation to the several states is to be made in the following manner: One-third in the ratio of the area of the state to the total area of the United States; one-third in the ratio of the state's population to the population of the United States, and one-third in the ratio of the mileage of rural delivery routes in the state to the total. In order that a state may obtain the fund to which it is entitled, its legislature must assent to the provisions of the act and the State Highway Department must submit to the Secretary of Agriculture projects for the construction of rural post roads, with such surveys, plans, specifications, and estimates therefor as he may require. "The Secretary shall approve only such projects as may be substantial in character." Items for engineering, inspection and unforeseen contingencies must not exceed 10% of the total estimated cost. The proportion of the cost of the work payable from the Federal treasury must not exceed 50%. No payment shall be made in excess of $10,000 per mile, exclusive of the cost of bridges of more than 20 ft. clear span. Responsibility for maintenance of the roads so built is placed upon the state or its "civil subdivision"; and if any such roads are not properly maintained, the Secretary of Agriculture must refuse to grant further Federal appropriations to the delinquent state or section. The act further appropriates $1,000,000 for the fiscal year ending June 30, 1917, for the survey, construction and maintenance of roads and trails within or partly within the national forests. All work done by the Government under this act is to be carried on by employees selected from the eligible lists of the Civil Service Commission.

PERSONALS

G. L. Marick has been appointed Assistant Office Engineer of the Gulf, Colorado & Santa Fe Ry., at Galveston, Tex.

James Wall, of Pittsburgh, Penn., has been appointed a member of the County Engineer's staff of Allegheny County, Pennsylvania.

Harry Burgess, Major, Corps of Engineers, U. S. A., has been detailed to succeed Col. Mason M. Patrick, in charge of the Detroit, Mich., district.

John C. Oakes, Major, Corps of Engineers, U. S. A., has been detailed to succeed Col. George A. Zinn, in charge of the Philadelphia, Penn., district.

A. E. Owen, Assoc. M. Am. Soc. C. E., Principal Assistant Engineer of the Central R.R. of New Jersey, has been promoted to be Chief Engineer, succeeding the late J. O. Osgood.

William V. Judson, Lieutenant-Colonel, Corps of Engineers, U. S. A., has been detailed to succeed Col. John Biddle, in charge of the Baltimore, Md., district.

George P. Clay, an inspector in the fourth district of the Pennsylvania State Highway Department, at Franklin, Penn., has been promoted to be Road Superintendent of Erie County.

A. Engh has been appointed Assistant Bridge Engineer of the Chicago, Burlington & Quincy R.R. Lines East of the Missouri River, succeeding G. A. Haggander, promoted, as noted elsewhere.

R. M. Robinson, Jun. Am. Soc. C. E., has resigned his position with the Tennessee Copper Co., Copperhill, Tenn., to join the staff of the Clinchfield Products Corporation, Johnson City, Tenn.

B. B. Weinberg, an inspector in the 14th district of the Pennsylvania State Highway Department, at Washington, Penn., has been promoted to be Road Superintendent of Blair and Cambria Counties.

Judson R. West, Assoc. M. Am. Soc. C. E., has resigned as Chief Engineer of the Seattle (Wash.) Port Commission to accept the professorship of railroad engineering at Pei Yang University, Tientsin, China.

W. H. Franklin, Jun. Am. Soc. C. E., has resigned from the valuation department of the Chicago, Milwaukee & St. Paul Ry., at Seattle, Wash., to join the staff of the James Black Masonry and Construction Co., Seattle.

G. A. Haggander, Assistant Bridge Engineer of the Chicago, Burlington & Quincy R.R. Lines East of the Missouri River, has been promoted to be Bridge Engineer of the whole system, succeeding the late C. H. Cartlidge.

Thomas A. Laux, Assistant to the Chief Engineer of the Pennsylvania Steel Co., Steelton, Penn., is now Assistant to the Chief Engineer of the Bethlehem Steel Co., South Bethlehem, Penn. Mr. Laux is a civil engineer of Harrisburg, Penn.

J. B. Chapman, a graduate physician of Alabama State University and a former student in the course in public health at Harvard University, has been appointed Health Officer of Talladega County, Ala., at a salary of $3,000 per annum.

Henry O. Pond, M. Am. Soc. M. E., has resigned as Mechanical Engineer of Westinghouse Church Kerr & Co., New York City, to take charge of the timber, pulp and water-

power interests in the Northwest of E. B. Cadwell & Co., Inc., New York City.

A. N. Wardle and **Ellis S. Tisdale,** both recent graduates of the course in sanitary engineering, Massachusetts Institute of Technology, have been appointed members of the engineering staff of the West Virginia State Department of Health, at Charleston, W. Va.

J. P. Stevens, recently General Superintendent of the Chesapeake & Ohio Ry., at Huntington, W. Va., has been promoted to be General Manager with headquarters at Richmond, Va. He is a son of George W. Stevens, President of the Chesapeake & Ohio; he is 31 years old and has been in the employ of the railway since 1900.

John Bennett Bissell, formerly President of John Bennett Bissell, Inc., exporters, has resigned to join the organization of the Blaw Steel Construction Co., Pittsburgh, Penn. Mr. Bissell will be Manager of the Export Department, and the Eastern Structural Sales Department, and will be located at the company offices, 165 Broadway, New York City.

Fred C. Alber, City Manager of Newburg, N. Y., has been removed by the city commissioners. He was formerly City Park Commissioner of Cleveland, Ohio. He was made City Manager Jan. 1, and according to a Cleveland newspaper account, has had a hard time from the start, chiefly because he was an outsider and because his salary of $5,000 per annum was considered too much. He is succeeded by a local physician, Dr. Henry Wilson.

Thomas C. Desmond, Assoc. M. Am. Soc. C. E., Consulting Engineer, 110 West 34th St., New York City, has been selected by Col. Theodore Roosevelt to take charge of recruiting the regiment of engineers, which is to be included in the colonel's army for Mexican or other service. Col. Laurance H. Grahame, formerly of the Rough Riders, is coöperating in the work of organizing the engineer regiment. Further data may be secured by addressing Mr. Desmond.

Frank S. Krug, City Engineer of Cincinnati, Ohio, and Chief Engineer of the City Rapid Transit Commission, will receive one of the largest salaries paid to an American municipal engineer. As City Engineer he receives $6,000 per annum, and as Chief Engineer of the Transit Commission he has been voted $9,000 more, making $15,000 per annum. Alfred Craven, Chief Engineer of the Public Service Commission, New York City, receives $20,000, which is probably the largest salary paid any municipal engineer in this country.

L. B. Allen, Division Superintendent of the Chesapeake & Ohio Ry., at Huntington, W. Va., has been promoted to be General Superintendent to succeed J. P. Stevens, promoted, as noted elsewhere. Mr. Allen is a graduate of Kentucky State College and began his railway work as a rodman on the Southern Ry. in 1899. He was with the engineering department of the Chesapeake & Ohio from August, 1899, until two or three years ago, when he joined the operating staff as a division superintendent. He had previously been Engineer of Maintenance-of-Way.

OBITUARY

H. J. Bowman, M. Can. Soc. C. E., of Bowman & Connor, Consulting Engineers, Berlin, Ont., died recently. He was a former city engineer of Berlin.

F. A. Creighton, M. Can. Soc. C. E., a consulting engineer of Winnipeg, Man., former city engineer of Prince Rupert, B. C., was recently killed in action in France. He had shortly before received the cross of the Legion of Honor from the French Government for bravery at the front.

Joseph Ramsey, M. Am. Soc. C. E., President of the Lorain, Ashland & Southern R.R., former President of the Wabash R.R., died July 7 at his home in East Orange, N. J. He was born at Pittsburgh, Penn., Apr. 17, 1850. He was educated at the Western University of Pennsylvania and in 1869 joined the engineer corps of the Pittsburgh, Cincinnati, Chicago & St. Louis Ry. For many years he was active in railway construction in Pennsylvania and Ohio, serving as chief engineer of small roads which were subsequently absorbed by the larger systems. He was Chief Engineer on the location and afterward Chief Engineer on construction of the famous Bell's Gap R.R. (from Bellwood to Irvona, Penn., now a part of the Pennsylvania R.R.) From 1886 to 1890 Mr. Ramsey was Chief Engineer of the Cincinnati, Hamilton & Dayton Ry. For a year following he was Assistant to the President of the Cleveland, Cincinnati, Chicago & St. Louis (Big Four) Ry., and then for two years General Manager. He was Vice-President and General Manager of the Wabash R.R. from 1895 to 1897 and subsequently was President of this and subsidiary lines. While President of the Wabash Mr. Ramsey was sent to the Paris International Exposition as Railroad Commissioner of the United States. Since 1913 he had been President and General Manager of the Lorain, Ashland & Southern R.R. in Ohio, of which he supervised the construction.

ENGINEERING SOCIETIES

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Aug. 29-Sept. 1. Convention in Providence, R. I. Secy., James McFall, Roanoke, Va.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 4-7. Convention in Hot Springs, Va. Asst. Secy., E. A. Baily, 360 Pearl St., Brooklyn, N. Y.

THE TRAVELING ENGINEERS' ASSOCIATION.
Sept. 5-8. Annual meeting in Chicago, Ill. Secy., W. O. Thompson, care General Offices, N. Y. C. R.R., Cleveland, Ohio.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
Week of Sept. 11. Annual meeting in Cleveland, Ohio. Secy., A. O. Backert, 12th and Chestnut St., Cleveland, Ohio.

THE AMERICAN INSTITUTE OF METALS.
Week of Sept. 11. Meeting in Cleveland, Ohio. Secy., W. M. Corse, 106 Morris Ave., Buffalo, N. Y.

NATIONAL ASSOCIATION OF STATIONARY ENGINEERS.
Sept. 11-16. Convention, Minneapolis, Minn. Secy., Fred W. Raven, 417 S. Dearborn St., Chicago.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 12. Convention in Atlantic City. Secy., A. P. Dane, Reading, Mass.

RAILWAY SIGNAL ASSOCIATION.
Sept. 12-14. Convention in Mackinac Island. Secy., C. C. Rosenberg, Myers Building, Bethlehem, Penn.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 18-20. Convention in Philadelphia, Penn. Asst. Secy., C. D. Fawcett, 29 West 39th St., New York, N. Y.

ASSOCIATION OF IRON AND STEEL ELECTRICAL ENGINEERS.
Sept. 18-22. Convention in Chicago. Secy., W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

THE ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 19-22. Convention in New York City. Secy., P. J. McAndrews.

MICHIGAN GAS ASSOCIATION.
Sept. 21-22. Annual meeting in Detroit, Mich. Secy., Clark R. Graves, Lansing, Mich.

AMERICAN PEAT SOCIETY.
Sept. 21-23. Annual meeting in Washington, D. C. Secy., Julius Bordollo, Kingsbridge, N. Y.

RAILWAY FIRE PROTECTION ASSOCIATION.
Oct. 3-5. Convention in New York City. Secy., C. B. Edwards, Mobile & Ohio R.R., Mobile, Ala.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9-13. Convention in Newark, N. J. Secy., Charles C. Brown, Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Convention at Atlantic City, N. J. Secy., E. B. Burritt, 8 West 40th St., New York City.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Convention in New Orleans. Secy., C. A. Lichty, C. & N. W. Ry., Chicago.

AMERICAN GAS INSTITUTE.
Oct. 17-20. Annual meeting in Chicago. Secy., G. G. Ramsdell, New York, N. Y.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Oct. 24-27. Convention in Cincinnati, Ohio. Secy., Prof. Selskar M. Gunn, Boston, Mass.

The Pacific Coast Gas Association will hold its 24th annual convention at Santa Barbara, Calif., Sept. 19 to 22.

The Pacific Association of Consulting Engineers holds its annual meeting in San Francisco, on Sept. 21. The secretary is C. H. Snyder, 251 Kearny St., San Francisco.

The Fifth Annual Safety Congress will be held by the National Safety Council, Oct. 16 to 21, in Detroit, Mich. The secretary is W. H. Cameron, Continental and Commercial Bank, Chicago.

The American Society of Mechanical Engineers, through the nominating committee, has reported the following nominees: For president, Ira N. Hollis, Worcester, Mass.; for vice-president for two years, Charles H. Benjamin, Lafayette, Ind.; Arthur M. Greene, Jr., Troy, N. Y.; Charles T. Plunkett, Adams, Mass.; for manager for three years, Robert H. Fernald, Philadelphia, Penn.; William B. Gregory, New Orleans, La.; C. R. Weymouth, Berkeley, Calif.; for treasurer, William H. Wiley, New York. The nominating committee was appointed by the president from five groups of the different geographic sections of the society, and consisted of Walter B. Snow, chairman, representing the Boston, New Haven and Worcester sections; H. M. Montgomery, representing the Chicago, Milwaukee and Minnesota sections; E. H. Ohle, representing the Atlanta, Birmingham, St. Louis and Cincinnati sections; J. T. Whittlesey, representing the San Francisco and Los Angeles sections, and D. Robert Yarnall, representing the New York, Philadelphia and Buffalo sections.

Appliances and Materials

Lubricator for Elevator Guide Rails

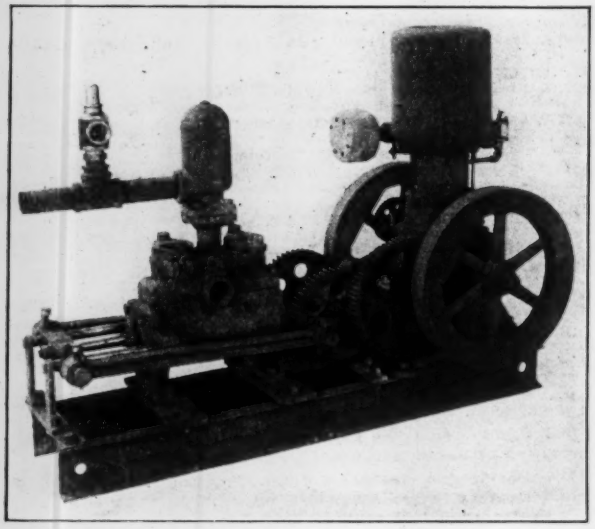
An elevator-rail lubricator of the type shown herewith is manufactured by Lewis F. Lyne, 39 Cortlandt St., New York City. It is an oil box cast from aluminum alloy and is ⅛ in. thick. Its only movable parts are three pairs of wicks intended to prevent glazing or caking over the surfaces. The wicks wipe off the edges and sides of the rails and at the same time deposit oil upon the cleaned surfaces. The feed is positive, no oil being delivered except when the car is running. The wicks are held by screws that can be loosened to allow the wicks to be pulled out a little when they become worn. Each box has a suitable support or bracket for attaching it to various types of elevator cars. It can be fastened to the counterweights as well as to the car proper.

* * *

Contractors' Pressure Pump

A high-pressure pumping outfit for use on construction work, delivering water considerable distances to supply boilers, concrete mixers, etc., is shown in the accompanying view. It consists of a single-cylinder vertical gasoline engine, gearing, and horizontal double-acting force pump mounted on a steel frame or bedplate as a single unit. The countershaft has two gearwheels carrying side rods which extend along the sides of the pump cylinder and are attached to the crosshead. These gearwheels are carried in an extension of the pump casting, so that the alignment is maintained permanently. The crosshead slides on guide rods attached to the cylinder head. There is a suction opening on both sides, and the discharge can be made at four points. The pump can deliver to a distance of three miles, and will work on a suction lift not exceeding 25 ft. It is made for pressures of 175, 225 and 300 lb.

The small-size outfit has a pump cylinder 3x5 in., 1½-in. suction and discharge openings, and runs at 55 r.p.m. It has



PORTABLE PRESSURE PUMP FOR CONTRACTORS

a 3-hp. engine, and can deliver 900 gal. per hr. A larger machine with 5x5-in. cylinder, 2½-in. pipes and 8-hp. engine can deliver 2,400 gal. per hr. The weight of the outfit is only 625 lb. for the smaller and 1,635 lb. for the large size. These portable pressure pumping outfits are built by the Barnes Mfg. Co., of Mansfield, Ohio.

* * *

"Manograph" Differential Pressure Gage

The "Manograph" recording differential pressure gage devised by Paul Lanham, of Washington, D. C., for use on the water-leakage surveys of that city and described in "Engineering News," Jan. 6, 1915, is now being made by the Water Works Equipment Co., 50 Church St., New York City. New models have been added to the line—notably (1) an instrument for permanent station installation, and (2) a multiple-unit recorder for measuring the flow through a number of mains. It will be recalled that the Lanham scheme involves a mercury U-tube, a moving contact rod actuated by a water or other motor and an electric circuit controlling the recording pen.

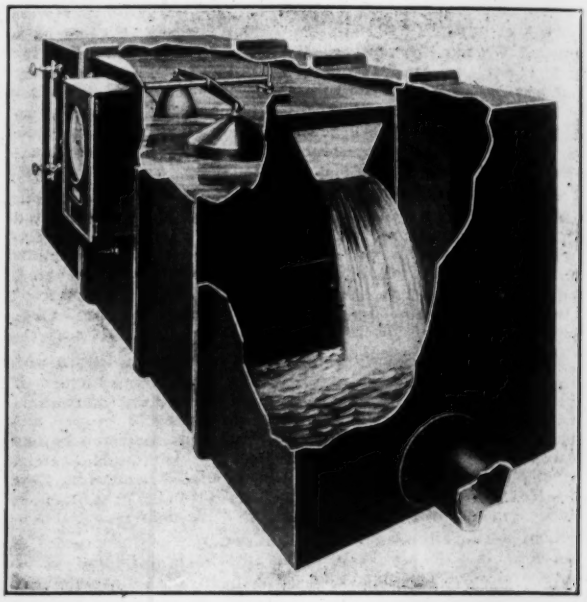
Dry Feeder for Water-Works Chemicals

A new dry feeder for lime, iron, soda ash, crushed alum, etc., in water-purification or softening plants is being made by the F. G. Gauntt Manufacturing Co., Fort Wayne, Ind. Cast-iron ends support a trough in which a worm agitator and feed screw revolve. The amount of material to be discharged is controlled by changing the length of an adjustable reach rod driving the feed. A machine with bronze worm and wood trough is also made for dryfeeding hypochlorite of lime, delivering from 4 to 200 lb. per hr.

* * *

Improved Weir Flow Meter

A new flow meter for feed-water tanks, etc., has been designed by E. G. Bailey, President of the Bailey Meter Co., 141 Milk St., Boston, Mass. As shown in the accompanying view, water flows over a V-notch weir from a high-level to a low-level compartment. Interest mostly centers on the mechanism for giving the recording-pen arm uniform motion



BAILEY RECORDING WEIR METER

proportional to the flow. A beam carries two unlike displacement members, and is fastened to a short shaft supported on monel-metal knife edges. One end of the shaft passes through a pressure-tight bearing and carries the recorder pen and integrator control directly on its outer end. The integrator disk is driven by the same clock that drives the chart; the counting train is suspended from a knife-edge bearing on the clock frame and is moved across the disk by an arm from the main shaft. The follower wheel has a large number of rollers to prevent wearing the disk, and its axis is inclined to eliminate slip.

* * *

New Section of Lackawanna Steel Sheetpiling

A new Lackawanna 8½-in. section of steel sheetpiling has been added to this line, to give a wider range of selection. This new section is of the standard Lackawanna straight-web type, with interlock consisting of similar hooks and guards on both edges, and is for use where a comparatively light section is required with high transverse strength and minimum weight—as in medium trench and coffer-dam work. The general advantages of the Lackawanna design are well known. In the development of this new section special attention was given to secure for this joint high tensional and transverse strength and at the same time to produce a pile that would have comparatively low weight per square foot, that would be simple, interlock perfectly and have the material so distributed as to give a high section modulus per lineal unit of wall.

The new section weighs 14.70 lb. per lin.ft. and 20.75 lb. per sq.ft. The least radius of gyration is 0.509 and the section modulus 1.1 per single section, 0.1294 per in. width and 1.555 per horizontal foot of wall interlocked in place. This section is to be compared with the older 7¼-in. piling—least radius of gyration 0.373; modulus of section, 0.567 for single section, 0.081 per in. width; weight, 12.54 lb. per lin.ft. and 21.5 lb. per sq.ft.

Lackawanna steel sheetpiling is made by the Lackawanna Steel Co., Lackawanna, N. Y.